In this issue:

*Hail, Caesar—
as Lauren leads applause
for AWWA's new President;
Bal Harbour, May 19*

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WATER WORKS ASSOCIATION

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Coming Meetings

AWWA SECTIONS

Sep. 7–9—South Dakota Section, at Grand Hotel, Watertown. Secretary, J. Darrell Bakken, Div. of San. Eng., State Dept. of Health, Pierre.

Sep. 12–14—Kentucky-Tennessee Section, at Andrew Johnson Hotel, Knoxville, Tenn. Secretary, Harold F. Mount, Gen. Mgr., Preston Street Road Water Dist. No. 1, 5400 Preston Hwy., Louisville, Ky.

Sep. 14–16—Virginia Section, at Cavalier Hotel, Virginia Beach. Secretary, Edward H. Ruehl, R. Stuart Royer & Assoc., 15 W. Cary St., Richmond.

Sep. 14–16—New York Section, at Saranac Inn., Upper Saranac Lake. Secretary, Kimball Blanchard, New York Branch Sales Office, Neptune Meter Co., 22-22 Jackson Ave., Long Island City 1.

Sep. 21–23—Michigan Section, at Park Place Hotel, Traverse City. Secretary, T. L. Vander Velde, Chief, Section of Water Supply, State Dept. of Health, DeWitt Rd., Lansing.

Sep. 21–23—North Central Section, at Lowry Hotel, St. Paul, Minn. Secretary, Carl A. Flack, Registrar, Water Dept., 216 City Hall, St. Paul, Minn.

Sep. 25–27—Missouri Section, at Governor Hotel, Jefferson City. Secretary, Warren A. Kramer, Chief, Water Supply, Div. of Health, State Office Bldg., Jefferson City.

Sep. 28–30—Wisconsin Section, at Loraine Hotel, Madison. Secretary, Harry Breimeister, Bureau of Engineers, 607 Municipal Bldg., Milwaukee.

Sep. 28–30—Canadian Section, Maritime Branch, at Charlottetown Hotel, Charlottetown, P.E.I. Secretary, J. D. Kline, Gen. Mgr., Public Service Com., Halifax, N.S.

Oct. 6—Connecticut Section, at Sanford Barn, Hamden. Secretary, Donald W. Loiselle, Supt. of Supply, Bridgeport Hydraulic Co., Bridgeport.

Oct. 6–7—Intermountain Section, at Newhouse Hotel, Salt Lake City, Utah. Secretary, M. W. Snell, Supt., Power & Light Dept., Soda Springs, Idaho.

Oct. 9–12—Alabama-Mississippi Section, at Tutwiler Hotel, Birmingham, Ala. Secretary, Ernest Bryan, McWane Cast Iron Pipe Co., Box 2601, Birmingham, Ala.

Oct. 16–19—Southwest Section, at Galvez Hotel, Galveston, Tex. Secretary, Leslie A. Jackson, Mgr.-Engr., Municipal Water Works, Robinson Memorial Auditorium, Little Rock, Ark.

Coming Meetings————————————————————————

(*Continued from page 6*)

Oct. 19–21—Iowa Section, at Fort Des Moines Hotel, Des Moines. Secretary, J. J. Hail, Supt., Water Dept., City Hall, Dubuque.

Oct. 19–21—Chesapeake Section, at Sheraton Park Hotel, Washington, D.C. Secretary, Carl J. Lauter, 6955—33rd St., N.W., Washington, D.C.

Oct. 20–22—New Jersey Section, at Madison Hotel, Atlantic City. Secretary, A. F. Pleibel, Dist. Sales Mgr., R. D. Wood Co., 683 Prospect St., Maplewood.

Oct. 24–27—Rocky Mountain Section, at Broadmoor Hotel, Colorado Springs, Colo. Secretary, Harrison F. Kepner, Vice-Pres., Dana Kepner Co., 550 Alcott, Denver, Colo.

Oct. 25–27—California Section, at Lafayette Hotel, Long Beach. Secretary, F. F. Watters, Hydr. Engr., State Bldg., Civic Center, San Francisco 2.

Oct. 26–28—Ohio Section, at Deshler-Hilton Hotel, Columbus. Secretary, J. Howard Bass, Henry P. Thompson Co., 1720 Section Rd., Cincinnati.

Nov. 9–11—North Carolina Section. at Robert E. Lee Hotel, Winston-Salem. Secretary, T. Z. Osborne, Asst. Director of Public Works, Greensboro.

Nov. 13–16—Florida Section, at Galt Ocean Mile Hotel, Fort Lauderdale. Secretary, John G. Simmons, Plant Supt., Water Dept., West Palm Beach.

OTHER ORGANIZATIONS

Aug. 15–19—Course on "Recent Developments in Water Bacteriology," R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Write: Chief, Training Program, 4676 Columbia Pkwy., Cincinnati 26, Ohio (or to USPHS regional office).

Aug. 22–27—Seventh Internation Conference on Coastal Engineering, Kurhaus, Scheveningen, The Hague, Netherlands. Write: J. W. Johnson, Engineering Field Station, Bldg. 159, Univ. of California, Richmond 4, Calif.

Aug. 23–25—Symposium on water quality data collection and utilization, R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Write: Director, 4676 Columbia Pkwy., Cincinnati 26, Ohio.

Aug. 29–31—Symposium on water quality measurement and instrumentation, R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Write: Director, 4676 Columbia Pkwy., Cincinnati 26, Ohio.

Sep. 18–22—NEWWA, Queen Elizabeth Hotel, Montreal, Que.

Oct. 2–6—WPCF, Convention Hall, Philadelphia, Pa.

Oct. 10–14—Fall general meeting, AIEE, Chicago, Ill.

Oct. 17–21—48th annual National Safety Congress, Chicago, Ill., with sessions on industrial safety scheduled for Conrad Hilton, Pick-Congress, Sheraton Towers, Morrison, and La Salle hotels. Write: R. L. Forney, Secy., National Safety Council, 425 N. Michigan Ave.. Chicago 11, Ill.

Oct. 24–28—9th annual water works management short course, cosponsored by the Illinois and Indiana sections, AWWA, at Allerton Park, Monticello, Ill. Write: T. E Larson, Head, Chemistry Sec., State Water Survey Div.. Box 232, Urbana, Ill.

Nov. 27–Dec. 2—Annual meeting, ASME, Statler Hilton Hotel. New York, N.Y.

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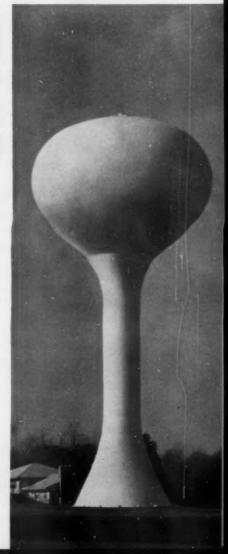
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#See Comparison Studies of Diatomite and Sand Filtration by G. R. Bell, Journal American Water Works Association, September, 1956 or write for free reprint.

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Economic Principles of Sound Main Extension Policies

Charles L. Stuart

*A paper presented on Oct. 30, 1959, at the California Section Meeting,
Bakersfield, Calif., by Charles L. Stuart, Vice-Pres., Southern Cali-
fornia Water Co., Los Angeles, Calif.*

IN the past 10 years, interest in the economic aspect of policies on main extensions has increased. The cause of the increased interest seems to be the result of an expanding population combined with a rising standard of living and consequent shifts in the location of population. This social ferment has, in turn, brought about a government policy of subsidizing the housing industry, presumably on the theory that housing is a matter of such broad political importance that this policy is a sound one. Many government housing projects have been built, and the government has made financing available to builders and developers under terms more favorable than would normally be called for. This has greatly stimulated the construction of new homes and is, perhaps, the largest single factor that has caused attention to become focused today on the economics of sound main extension policies.

Privately Owned Water Utility

Before the economic principles of sound main extension policies are examined, it is most pertinent to consider the economic characteristics of the privately owned water utility, its organization, the conditions of cost under which it operates, the nature of the demand for its services, its capital investment, the sources from which its capital funds are derived, and, to some degree, the sources and distribution of its utility income.

Organization. The corporation is the accepted form for the privately owned water utility. Economically, the advantages of the corporate form are especially significant. The relative permanence of the organization is highly important for a business that must endure as long as the public demand for its service continues. The large capital requirements of utilities render the limited liability feature of

particular importance in attracting adequate capital. The ability to issue a variety of securities—bonds, preferred stocks, and common stocks—permits an appeal to investors of divergent tastes. The provision for delegated management facilitates the concentration of the technical supervision of operations under the direction of those particularly competent in such matters.

Obligations. Privately owned utilities are distinguished from other businesses by the formal obligations to the public which are imposed on them. They must serve all who apply for service in their area, and must charge customers reasonable and nondiscriminatory rates. They must be prepared at all times to render a service that provides a product adequate both in quality and quantity, and, further, this service must be immediately available when the customers demand it.

Capital structure. The fixed public obligations imposed on privately owned utilities have resulted in a capital structure characterized by a large investment in utility plant per customer—large, that is, by comparison with other types of enterprises. This large investment per customer arises mainly from the obligation of the utility to supply immediately the customers' demands for service, and, in addition, meet the community's fire service needs. The water is not consumed steadily throughout the day and is only storable at selected locations. The result is much unused plant capacity when demands are at a minimum. The mains, pumps, and storage capacity needed for maximum consumption and the community's needs for fire service are items that require large sums of money to construct. And, finally, the necessary investment is made even greater by poor seasonal and annual load factors in the water supply industry. Investments have also been multiplying as a result of rapid population growth.

Capital market. The large capital investment per customer characteristic of privately owned utilities, combined with population growth, has made the availability of new capital a matter of critical importance. If utilities are to fulfill their public responsibilities, it is essential that an adequate supply of new capital not only be available but also be available on the most favorable terms that the market will afford. Under normal circumstances, utility companies have to depend on the sale of new securities to provide required capital. The regulation of utility earnings has not made it possible for these companies to provide for their capital requirements through reinvestment of earnings. In this respect also, the operations of privately owned utilities contrast with operations in other industries and trades.

The capital market, to which privately owned utilities must go for their funds, has more elements of competition than do most other markets. In the money market, there is no differentiation of the product, because all dollars are alike. These dollars, moreover, are free to move at will to the location of highest yield. The managements that will use the funds are considered in terms of their skill and their ability to produce a profit. The presence or absence of regulation is considered. If regulation is present, its historical and philosophic approaches are evaluated. The decision as to what company gets money, the amount and the cost, is made on an unemotional and logical basis. Although there are elements of government control in the capital market,

such controls are there to strengthen the forces of competition rather than to soften them. Thus, one of the distinguishing characteristics of the economics of a privately owned water utility is an unusually high investment per customer—an investment that has to be procured against competition in the public money market, because the amount of surplus accumulated out of income is small.

Annual turnover. Another significant economic characteristic of privately owned utilities is revealed by the "annual turnover" or the relationship of the operating revenues of these companies to their total capital. In contrast to other forms of business enterprises, water utilities have a low annual turnover—that is, their operating revenue is a relatively small percentage of their total invested capital when compared with revenue of other business enterprises. It typically requires 6–7 years for a privately owned utility to earn gross revenues equal to its total invested capital. At the other extreme, trading corporations frequently have yearly revenues that are four to five times their invested capital.

Economics of Different Utilities

At this point, it is important to mention a subject frequently brought up in discussions about financing main extensions. Almost invariably, the builder will ask, "Why doesn't the water company follow the same financing policies that the gas, electric, and telephone companies follow in making service available to my subdivision?" From this question, it is apparent that the different economics inherent in different types of utilities are not recognized. This lack of understanding also exists in some regulatory quarters where the desire seems to be to apply

the same treatment for main extension policies to all of the different types of utilities.

Annual turnover. With regard to gas utilities, the annual turnover is generally 2.5 years; in the electric industry, approximately 4 years; and in the telephone industry, generally 3 years. These turnover times are approximately one-half that for water utilities. One of the most important aspects of the better turnover for other utilities is that it gives them a substantially better opportunity to counteract the erosive effect that inflation has on their capital investment.

Size. Another important difference between water utilities and other utilities is size. Water utilities are generally smaller, local utilities whose securities are not often traded nationally and whose credit does not have the wide market available to the larger utility organization, typical of other utility fields. Furthermore, per customer, the use of water has not increased as much as the use of gas, electricity, and telephones. This means that water utilities, as compared to other utilities, have not had as much benefit from decreasing unit costs usually associated with increasing per capita consumption.

Load factors. Again, water utilities are characteristically different from other utilities because they have substantially poorer annual and seasonal load factors. This deficiency has an adverse effect because it requires a larger capital investment in facilities only used for short periods of time. In this connection, too, water utilities are unique in that the additional capacity for community fire protection has to be added to peak customer demands.

Cost of service. Another distinguishing factor among utilities is the cost of rendering service. Water utili-

ties have not shown progress in decreasing the costs of production, as have electric utilities; have not improved in mechanization, as have communications utilities; and have not developed new lower-cost sources of supply, such as the supply of natural gas developed by the gas industry. All other utilities have had some far-reaching developments that have kept down the cost of service in an era of steadily rising prices.

Depreciation charges. The economics of water utilities are substantially different from the economics of gas, electric, and telephone utilities with regard to depreciation charges. These charges, based on the historical cost of property, tend to be inadequate. It follows that, when such charges are insufficient to replace property at the end of its useful life, an additional erosion of the utility's investment has taken place. For utilities in the water supply field, one deals with long-lived materials that are depreciated over longer periods of time than is true for utilities in other fields. In other words, water utilities have a slow rate of capital substitution. This means that inflation has a long time in which to undermine the real value of the depreciation accruals. When these foregoing economic facts are considered together with the strict regulation of utility earnings, a potentially dangerous situation is apparent. Certainly, with present methods of depreciation accounting, it will be necessary, if inflation continues, for water utilities to keep raising additional new capital from outside sources just to maintain a fixed amount of physical property in being.

Operating ratio. The low turnover characteristic of water utilities makes it essential that a large portion of each dollar of earnings be available to pay a return on invested capital. The earnings available for the payment of return on capital depend on the relative magnitude of operating expenses. The relationship between operating expenses and operating revenues is expressed generally as "the operating ratio"—the ratio of the operating expenses, including taxes and depreciation, to revenues. The lower the operating ratio, the larger is the proportion of income available to pay interest and dividends on outstanding securities. Water utility operating ratios vary rather widely with topography, the degree of saturation of customers and their usage characteristics, climate, whether or not there is a local source of water or a distant one, whether the utility produces all of its own water supplies or has to purchase substantial quantities, whether there is one central source or many scattered sources, and other factors.

Main Extension Policies

The foregoing broad picture of the economic characteristics of privately owned water utilities should aid in the understanding of economic principles for sound main extension policies. These are necessary to help maintain the healthy, economic climate needed for a utility to fulfill its obligations properly.

Historical aspect. It is unwise to make radical changes in main extension policies which might serve to upset significantly the economics evolved under the application of a given rule. Any rule, however, should be reviewed periodically, and, although radical changes should not be made, progress toward worthwhile objectives may be brought about in a gradual and orderly manner.

Objectivity. A sound main extension policy should be objective. It should be devised only after the customer, the investor, and the community are considered.

Rates of existing customers. Evidence indicates that in most states—and in California in particular—regulatory policy is to keep the rates the customer pays for water service as low as possible. It is therefore reasonable to assume that the public utility commissions would be concerned with the effect that any extension policy would have on the rates of existing customers. No main extension policy should require a company to make an investment so much greater than its present average investment per customer that the rates of the existing customers will be increased. Existing customers have, in the first place, made it possible for new customers to obtain water service. It is these existing customers who have, over the years, made possible the development of the water plant and distribution system that will serve as the foundation for the extension of service to new customers. In this regard, it should not be overlooked that the utility makes the investment in the cost of the water supply, the general property, the large mains used in the general supply system, and the distribution storage. A sound policy must consider that an increase in the number of customers requires additions to general plant all along the line, over and above the facilities for immediate service to the development being considered.

Protection against speculation. Any sound main extension policy should protect the utility against speculation. In recent years, most building projects have been complete projects—that is, homes were built in conjunction with the subdivision of the land and the lots.

Because of rising land prices in the past several years, the practice is returning of selling lots only, without building homes in conjunction with the development. Also significant is the comparatively recent phenomenon of "industrial subdivisions." These, too, in most instances, are for the sale of fully developed lots only, but are distinguished by the use of large mains and large sewers, and represent a large future liability for service facilities in the supply of consumptive and fire protection needs.

Financing. A sound main extension policy should consider the necessity of preserving the financial ability of the utility and should not, by its terms, limit the financing methods and financing procedures that can be followed by the utility. Utility managements need to use ingenuity, even innovation, in raising capital, because of the handicaps imposed on them by the nature of their business, the economic conditions affecting the business, and because of regulations. Any policy should allow for the flexibility needed to insure that utility managements will be free to choose the best available means and methods of handling financial problems.

Replacement of facilities. As stated earlier, a sound main extension policy should make allowance for a more realistic depreciation charge. Plans have been suggested which were based on periodic adjustments of the depreciation charge made on the basis of changes in construction cost indexes; such plans may be feasible.

The utility account system prescribed by the California Public Utilities Commission generally furnishes the factual material on which the regulatory process operates in its effort to substitute for the absence of competition in fixing prices and production of all utility

services. The fundamental premise of the depreciation accruals under the prescribed accounting system seems to be that over the years the dollar has a stable purchasing power. This premise was probably valid back in the days when the United States was on the gold standard. With respect to the dollars recorded for utility plant depreciation expenses in the years after World War II, the premise no longer holds true. The objective of making depreciation provisions is to maintain intact the capital assets of the enterprise. This means replacing productive capacity as it wears out, not just recapturing a fixed number of dollars.

The two quantities in which the average investor is primarily interested are the return on his investment and the preservation of its value. It is this preservation that requires proper depreciation accounting to provide for a complete replacement of the utility plant at the end of its life. The fact that water utilities turn over their capital only once every 6 or 7 years, in contrast to a turnover of as much as four or five times a year for other types of businesses, puts the utilities at a great disadvantage in the capital market in a time of continuing inflation.

In stating that a sound main extension policy should give the problem of plant replacement consideration, the reasoning is as follows: The investment in main extensions necessary to serve rapid growth is an obligation that the utility has to meet as a part of its responsibility. In other words, there is no managerial discretion as to whether or not the utility shall assume the liability for replacing the facilities necessary to serve the growth in its territory. If the inflationary trend of the past 20 years continues, it is evident that when the time comes to replace facilities the

depreciation accrual will be insufficient. It will then be necessary for existing customers of the utility to pay the costs of borrowing the additional moneys necessary to replace these facilities. This will again increase the rate base and again make rate adjustments necessary. An attempt should be made now to halt this continuing spiral.

Competition. A sound main extension policy should insure that a regulated utility can compete with an unregulated utility in situations where such competition is essential to the effective operation of the regulated utility and, therefore, to the well-being of its customers. Such insurance can be furnished by a provision for the use of proportionate cost refunds *at the utility's option.*

Investments. In determining how much of the cost of the extension should be advanced by the builder, a sound policy should recognize that the utility makes an immediate investment in the water supply, storage, transmission mains, and meters. The general ratio of such investments to advances is approximately $1 invested by the utility for every $2 advanced by the builder.

Market rates. A sound policy should also permit the discounting of builders' advances at current market rates, not at an artificial fixed rate. Situations exist where people who make no contribution to a solution of water supply problems (and who are not associated with a water utility) make large profits simply by buying builders' main extension contracts at prices well below those that the regulated utilities are allowed to pay and then by collecting their profits from the utility refunds.

Engineering specifications. A sound policy should recognize that the builder-

subdivider, by the nature of his operation, takes a short-term view. He is in and out of the typical subdivision within a year's time. The money he advances on a main extension contract will be charged against the houses, and he will recoup this money in the sale price of the home. He will then collect this money a second time from utility refunds. The utility, however, assumes the obligation of service to the development for perpetuity. This disparity of viewpoint is a sound reason why the choice of materials, the sizing of pipelines so that orderly future growth can take place without future charges, and other engineering specifications have to be left to the discretion of the utilities. Also to be considered is the utility's right to make further extension without incurring additional obligation to the original extension promoter. In this respect, it has been noted that new customers in the subdivisions frequently use more water than do older customers. This may be the result of landscaping, other requirements of new construction, and, in part, new equipment, such as garbage disposal units, dishwashers, sprinkler systems, and air conditioners. The result of this increased demand is that new customers require larger feeder mains, larger tract mains, more pump capacity, more distribution storage, and a larger capacity for the source of supply. But the additional revenue derived from these new customers is not necessarily commensurate with their increased demands, because most water utilities use block charges, with prices declining as consumption increases. Moreover, many new tracts are at higher elevations that require extra pressure for water delivery, additional storage, and duplication of pumps to guard against failure. All in all, the utilities' liability of service to these newly developed areas is a sizable one.

Refunds. As stated before, a sound policy would limit the refunds made to builder-developers to the average system investment in distribution mains, services, and hydrants. A larger refund has the effect of granting a subsidy to the builder-developer or providing him with more of a profit. It is no more logical for existing customers to subsidize the extension of water service than to subsidize the extension of streets, sidewalks, or sewers. Furthermore, refunds should not be based on revenue, because, to begin with, this implies that a utility takes money for investment in facilities from revenue, which is not true. The investment, for the most part, comes out of borrowings from the public market; revenues merely support the costs of paying the interest and dividends on this borrowing. In addition, when refunds are based on revenues, rate increases, usually due to many causes, speed up refunds. This accelerates the building up of the rate base, which, in turn, exerts additional leverage toward larger rate increases more quickly than would otherwise have been experienced. Finally, as discussed earlier, the operating ratios of water utilities vary widely because of differences in topography, source of supply, and amount of water that is purchased. Thus, a uniform percentage of revenue is not fair from this standpoint.

Fire protection. Certain advantages that builder-subdividers get from their dealings with a regulated utility should be given overall consideration in any main extension policy. One of these advantages is fire protection. This has a definite value to the subdivider because it gives him a good sales point when dealing with home buyers. Also,

fire protection unquestionably benefits the homeowner, for he saves a substantial amount on his insurance policies when water for fire protection is available.

Land values. Another advantage of dealing with a regulated utility is the increase in land value that results when such a utility, with established credit, service, facilities, water rights, and a staff of engineers and other specialists, is responsible for serving a development. In California, many instances have occurred where the value of land tripled and quadrupled because the owner made a contract for service with a recognized water utility.

Summary

A sound main extension policy must be based on economic facts pertaining to water utilities. Water utility economics are significantly different from the economics of other types of utilities. These differences warrant different main extension policies. Thus, a sound policy should:

1. Consider historical aspects
2. Be objective
3. Not tend to raise the rates of existing customers
4. Protect the utility against speculation
5. Preserve the financial ability of the utility
6. Consider the liability for replacement inherent in all new businesses
7. Provide a means to enable the regulated utility to compete with unregulated utilities
8. Recognize the utility investment in water supply, system storage, transmission mains, and meters
9. Permit the purchase of main extension contracts at current market rates
10. Leave the choice of materials, sizing, continuing extensions, and other engineering specifications to the utility
11. Base refunds on something other than revenue
12. Recognize the advantage of fire protection made available through utility services
13. Recognize the increase in land value that accrues when a recognized water utility pledges its water rights, service, and organization to a development.

Organization and Problems of the Fairfax County Water Authority

—James J. Corbalis Jr.—

A paper presented on Nov. 6, 1959, at the Virginia Section Meeting, Roanoke, Va., by James J. Corbalis Jr., Engr.-Director, Fairfax County Water Authority, Annandale, Va.

FAIRFAX County, Va., is an integral and increasingly important part of the metropolitan area of Washington, D.C. It is 400 sq mi in area and borders on the Potomac River, Arlington County, and the city of Alexandria, Va. In 1940, the population was 40,929; in 1950 the population had increased to 98,557; today, almost 10 years later, another more than twofold increase has brought the population to 232,000, and the latest forecasts of growth during the next 10 years promise a population of 417,500 in 1970 and 700,000 by 2000. The provision of adequate water service for this rapidly growing county will be an everyday problem for decades to come.

Present Diversities

At present three sources of water are used to serve the northern Virginia portion of the Washington metropolitan area. These are: the Potomac River, source for the District of Columbia water system and through which the city of Falls Church serves a substantial area in the northeastern part of the county; Occoquan Creek, source for the Alexandria Water Co., which serves a substantial area in the southern part of the county; and ground water, used by the systems serving the towns of Fairfax, Herndon, and Vienna and various other parts of the county. In addition, Fairfax is embarking on a program to develop a water supply from Goose Creek, located in adjoining Loudoun County. For the future, it is expected that the requirements of the northern half of the county, estimated to be 50 mgd, will be satisfied by water taken from the Potomac River and Goose Creek. The requirements of the southern half, also estimated to be 50 mgd, will be satisfied by water taken from Occoquan Creek. Ground water sources have generally failed to produce sustained yields and are not considered reliable enough to meet future needs.

In addition to the Fairfax County Water Authority, four municipalities and 15–20 privately owned companies supply water in the county. With this number of operating agencies it is normal to expect differences both in the quality of service rendered and in the charges made for it. In some instances there are serious deficiencies in the quantities and pressures available. For example, some systems provide domestic service only, whereas others provide varying degrees of fire protection, fire hydrants being installed either at the expense of the company or at the expense of the county, with their operation and maintenance provided by the county. There is no uniformity in kinds of pipe, valves, or hydrants. Interconnection

between systems is the exception rather than the rule. The desire or ability of these agencies to extend service beyond their immediate revenue-producing areas is subject to question. This is particularly significant in view of the large and often widely separated areas of the county which are subject to early development. For a consumption of 15,000 gal per quarter, the average charge varies from 40 to 90 cents per 1,000 gal. The service and rates of only seven of the approximately 25 agencies are under the control of the Virginia Corporation Commission.

Creation of the Authority

It was the diversity in water service and rates that prompted the board of county supervisors to create the Fairfax County Water Authority in 1957. The charter defines the authority's purposes as "the acquisition, construction, operation and maintenance of an integrated water system for supplying and distributing water in Fairfax County." The authority was created under the 1950 Virginia Water and Sewer Authorities Act. Although there now are nine other water authorities in various parts of Virginia, none of them are known to have been chartered for, or to be concerned with, the development of so comprehensive a program as the one in Fairfax County. There is almost no precedent for such action in Virginia, although, of course, a number of similar agencies have been successful elsewhere.

In Virginia, a water authority is a nonprofit, politic, and corporate instrumentality designed to exercise public and essential governmental functions. An authority may be created by one or more governmental units. Although established by government, an author-

ity is not ordinarily included in the normal structure of government. Its affairs are administered by a board of five members, or not less than one from each participating unit, who are appointed by the participating unit or units for terms not to exceed 4 years each. An authority is empowered: (1) to acquire, either by purchase, gift, lease, or, through the exercise of eminent domain, to construct, operate, and maintain any water supply system; and (2) to finance its programs through the issuance of revenue bonds without a referendum and, subject to the jurisdiction of the State Corporation Commission, to fix and prescribe rates for the service rendered. It cannot levy taxes or assessments, and the obligations of an authority do not become obligations of its participating units.

Acquisition of Annandale System

After a general assessment of the existing conditions and a definition of broad objectives, the Fairfax County Water Authority set itself to the task of getting into the water business. The county had been considering the purchase of the water system owned and operated by Annandale Water Co. This system, which served approximately 15,000 persons, was located adjacent to areas of prospective development and seemed well adapted to integration into any comprehensive system that might later be established. After many months of diligent effort, the authority concluded a contract to purchase this system. Among the problems encountered was the preparation of various studies in order to establish the fair market value of the system. This involved the usual considerations of original cost, reproduction cost, depreciation, obsolescence, and the earn-

ing power of the system, both at existing and at increased rates of charge. Because this system, like most of the privately owned systems in the county, included large amounts of capital donated by real estate developers who had installed facilities and subsequently conveyed them to the company without cost, there was strong public opinion that these so-called "windfalls" should be reflected in purchase price of the system. This also contributed to the idea that the system might be acquired through the exercise of eminent domain. Virginia had no legal precedent setting forth the effect of contributed capital in condemnation proceedings. A further complication was the owner's insistence that the stock, rather than the assets, of the company be acquired. This condition made necessary an estimate of the liabilities of the company, both known and unknown, before a purchase price could be established. Acquisition by an authority of the stock of a corporation is also without precedent in Virginia. Of equal financial importance was a determination of the immediate capital outlay required to eliminate deficiencies in the existing system and provide for the needs of an expanded service area. Finally, a purchase price of $1,225,000, plus $275,000 in estimated liabilities, and $1,500,000 for immediate capital outlays and other costs resulted in the sale of $3,000,000 in revenue bonds, and the authority entered the water business on Jan. 22, 1959.

In order to support this financing program the existing water rates had to be increased by approximately 30 per cent. The new rates are, however, about 5 per cent less than those requested in an application made by the Annandale Water Co. and pending before the State Corporation Commission. Furthermore, no plan had been announced for improving the system to the extent proposed by the new authority. Although the new rates are higher than anywhere else in the county, there is ample evidence that the achievement of a comprehensive system will bring lower rates than would otherwise prevail.

Northern-Area System

The next important activity in the brief history of the Fairfax County Water Authority was the assumption of a commitment to the federal government to supply water to the new Dulles International Airport, a jet plane terminal for Washington, D.C. The airport is now under construction on 9,000 acres located in the northwestern part of Fairfax County and extending into adjoining Loudoun County. Water is to be made available to the airport on or before Jan. 1, 1961. The commitment resulted from a review of the immediate and near-future needs of the northern area of the county, including the possible supplementing or supplanting of the present well supplies serving the incorporated towns, and a proposal for the construction of a $4,800,000 system.

Although it had been concluded earlier that the Potomac River was the most reliable source of water for the northern part of the county, inquiry made to District of Columbia officials resulted in assurances that the present and future needs of the authority could be supplied through the District of Columbia system. This meant that filtered water would be available at the district's treatment plant located on the opposite side of the river. (A future alternative, if it becomes economically

feasible, is the construction of a treatment plant on the Virginia side of the Potomac.)

Getting the water across the Potomac was a complex problem. River crossings are expensive in themselves, and, to add to the expense, sound engineering judgment dictates the selection of pipe of sufficient size to satisfy relatively long-term future requirements. Financial limitations, however, often make it necessary to deviate from the rule, as was the case with the authority. A crossing of sufficient capacity to meet the needs of 1970 was determined to be the minimum that could be considered, and even that was expected to cost about $500,000. Fortunately, Falls Church was completing a new 36-in. crossing of the river, and surplus capacity, enough for the authority's needs until about 1970, was expected to be available from this facility. After lengthy negotiations, an agreement was reached with the city of Falls Church whereby water will be furnished to the authority at cost, either at the river crossing or elsewhere throughout the system wherever and so long as a surplus capacity exists. This agreement will permit the authority to defer the construction of a separate river crossing for approximately 10 years, at which time the needs will be better defined and the authority's financial position should be more favorable. Meanwhile, Falls Church will also reap a benefit for it will be receiving payments for surplus capacity not otherwise useful to it. It is, however, significant that although both parties were assured of benefits through the joint use of the river crossing, the agreement was reached only after threats of litigation and Congressional action concerning the distribution of District of Columbia water in Virginia. This conflict developed because Falls Church had long served a substantial area of the county and had announced plans for serving a still greater area. This proposal jeopardized the authority's proposed operations. Although the rates charged by the city were not unreasonable, and the service was generally satisfactory, there was a strong feeling that Falls Church was profiting from the consumers in the county, customers who had no voice in the city government and for whom there was no other apparent relief. The controversy was settled by the delineation of a somewhat smaller area for the future expansion of the city system, within which the authority agreed to refrain from rendering service.

The authority's northern area system will have 24 mi of mains, two pumping stations, and elevated storage facilities—all capable of meeting needs expected during the period 1975–80. Detailed design, the receipt of construction bids, and the sale of revenue bonds are expected to be completed shortly. It is also expected that this program will include the acquisition of another of the privately owned water systems located in the northern part of the county.

Acquisition of Alexandria System

Another major activity during 1959 was consideration of joint acquisition by the city of Alexandria and the Fairfax County Water Authority of the Alexandria Water Co. system—the largest privately owned water system in Virginia. This system serves the city of Alexandria, a large portion of the southern part of Fairfax County, and a small portion of adjoining Prince

William County. It is intended that Alexandria will acquire and operate the distribution system within the city and that the authority will acquire and operate the production, transmission, and distribution facilities within the county. Water is to be furnished to Alexandria at cost.

After the Alexandria Water Co. had refused a $20,000,000 offer of purchase, the authority and the city each authorized the institution of condemnation proceedings to acquire those parts of the system located within the county and the city, respectively. Because acquisition of an existing water system by an authority through the exercise of the right of eminent domain, although the power is granted by law, has never before been attempted, the authority will again be pioneering. In fact, similar power granted to cities, although of much greater age and use, has not been exercised in Virginia in many years. Unlike the procedure that will be followed by the city of Alexandria, condemnation by the authority must be preceded by a finding of the State Corporation Commission that "a public necessity or that an essential public convenience shall so require." Thus, many interesting questions are expected to arise from the proceedings, and a new precedent will undoubtedly be established.

Future Acquisitions

Information on the character of the remaining privately owned systems is being assembled preparatory to their acquisition either by negotiation or by condemnation. Because of the number of systems involved and the difficulties of integrating them into a comprehensive system, this is hardly a task that can be completed quickly. It is similar to a jigsaw puzzle; piece-by-piece, the picture will be made whole.

Public Objections

The authority's announced intention of acquiring the privately owned system has not gone entirely without criticism. It has been contended that the action imperils the cherished free-enterprise system and that the customer will suffer financially.

The authority does not, however, have any aspirations toward "empire building" at the expense of private ownership; rather, it must resort to this action in order to achieve a comprehensive system free of the deficiencies in the present, independently operated systems and capable of meeting the needs of the tremendously expanding population. In so doing, the authority expects to compensate the present owners in a fair and just manner.

The cost of the program admittedly will not be small. The present value of the systems, including the large amounts of contributed capital in most of them, and the present high interest rates are significant factors. Offsetting these, however, are relief from local, state, and federal taxes and returns to stockholders. Assuming that the same or better service will be offered, the true test is not what initial increase in rates, if any, may be required, but whether, over a reasonable period of time, the rates will be lower under public ownership or under private ownership.

Merits of Authority Approach

Three features distinguish the authority from the usual form of public ownership, such as a municipal ownership:

1. The authority is a single-purpose, businesslike organization that can consider both day-to-day and long-range problems and may gear itself to find solutions as the circumstances warrant.

2. The authority possesses more liberal contractual powers, which are of particular value in the development of joint-use programs.

3. The authority can arrange its financing without express approval by a referendum.

Although these three features are desirable, it should be realized that any program undertaken must be financially feasible. The criterion for establishing this feasibility is more critical in authority revenue bond financing than in general-obligation bond financing. The element of risk involved in estimating future growth assumes greater proportion in revenue bond financing, whereas the full faith and credit of the community securing general-obligation bonds overshadows such risks. The absence of a requirement for referendum approval of the issuance of bonds is of special importance in situations similar to that in Fairfax County. The Fairfax County Water Authority's program is neces-

sarily a long-term one, and there will be wide variation in the problems encountered in each of its several parts. It is highly improbable that county-wide referendum approval would be assured on each part of the program, not because of any lack of merit, but because of the public disinterest or suspicion in areas not directly affected by the particular part of the overall program under consideration. Despite the lack of required referendum approval, the interests of the residents are safeguarded by the absolute requirement that revenue bonds cannot be issued unless the project can be financed through reasonable rates for the service to be rendered.

The authority approach to the problem of providing water for expanding suburban areas is by no means a panacea, but the experience of Fairfax County in the past 2 years shows that it can be successful. The authority is likely to be exposed to almost all of the difficulties which might be encountered in the development of a comprehensive water system, and it is hoped that at some future date Fairfax County can report new successes in the use of the authority approach.

1958 Federal Water Supply Act and Future Water Requirements

——Gordon E. McCallum and Robert W. Haywood Jr.——

A paper presented on May 19, 1960, at the Annual Conference, Bal Harbour, Fla., by Gordon E. McCallum, Chief, Div. of Water Supply & Pollution Control, USPHS, Washington, D.C., and Robert W. Haywood Jr., Director, Water Resources Sec., Region VII, Div. of Water Supply and Pollution Control, USPHS, Dallas, Tex.

THE water supply industry is working in an age of opportunity; it is coming to grips with some knotty problems. These problems differ somewhat from those faced, say, 50 or 100 years ago. More and more water is needed, but the industry must be conscious of quality as well as quantity. Not all water available in adequate quantity is of satisfactory quality. The opportunity to solve these problems is available; the tools and know-how are at hand.

Industrial and Population Growth

Most water men have felt the pressure of what has been termed "population explosion" and the feverish industrial activity that invariably accompanies it. Municipal and industrial water requirements have been growing by leaps and bounds. The estimates of United States water needs for the years 1975 and 1980 almost stagger the imagination. It is apparent that economic growth is quite closely related to the availability of adequate quantities of water. Furthermore, this water, to be of value, must meet rather rigid quality standards. Indeed, the industrial water quality requirements may become more rigid than those set for municipal usage. Those areas that are unable to keep pace with increasing demands for water may be left behind by the expanding economy.

The water supply industry, in general, has kept up in providing water in the quantity and of the quality required, but recently inadequacies have arisen. With the increased water demands being placed on existing sources of supply, many areas are experiencing water shortages where once there was plenty. In the year 2000, United States cities will require an estimated 58 bgd, in contrast to the 16.7 bgd supplied in 1954. The overall water supply problem is indeed a huge one, but when it is broken up into its parts, one can see that it is not beyond solving. Some of the legal tools necessary to get the job done are now available.

Federal Legislation

Until 1958, provisions for water supply storage could be included in Corps of Engineers projects under two authorities. The first of these, Public Law 208, which was approved by Congress on Jul. 19, 1937, authorized the modification of Corps of Engineers projects to include water supply storage if the states and political subdivisions would contribute funds to cover the increased cost for providing the

storage. The second, Public Law 534, approved by Congress on Dec. 22, 1944, and reenacted by Public Law 360 on May 23, 1952, authorized the Secretary of the Army to contract with states, municipalities, private concerns, or individuals, at such prices and terms as he deems reasonable, for any surplus water that may be available at a reservoir. This surplus water is made available for domestic and industrial use under this legislation.

Reclamation legislation in 1939 empowered the Secretary of the Interior to enter into contracts to furnish water for municipal water supply or "miscellaneous" purposes. The contracts are similar to those set up for the Corps of Engineers in that they require repayment of an appropriate share and spell out the terms and conditions of repayment, annual operation costs, and maintenance costs. It is further required that contracts do not impair the efficiency of the project for irrigation purposes.

Water Supply Act of 1958

In 1958, Congress enacted new legislation providing for inclusion of water supply storage in Bureau of Reclamation and Corps of Engineers reservoir projects. This legislation, Title III of Public Law 85-500, approved Jul. 3, 1958, is known as the Water Supply Act of 1958. In it, a general policy is provided for including water supply in civil works projects. It is applicable to new projects and modifications of previously authorized projects. The act affects only municipal and industrial water supplies. Also specified are the basis for determining the interest rate on unpaid balances—the responsibility of the Secretary of the Treasury—and other terms and conditions concerning inclusion of storage in

reservoir projects and repayment arrangements.

The Water Supply Act of 1958 is an expression of an important new federal policy. The provision authorizing the inclusion of storage for anticipated demands can be the key to providing for future water needs in many areas. The portions of the act that are applicable to this future demand storage include provisions that:

1. The allocated cost of such storage must not exceed 30 per cent of the overall project cost

2. Reasonable assurances must be obtained from state or local agencies that they will contract for use of the storage in time to permit full payment of the allocated costs within the life of the project, but not later than 50 years after the first use of the storage for water supply

3. No payment need be made for storage for future use until the storage is first used

4. No interest will be charged on the allocated cost of the storage until it is first used, but the interest-free period is not to exceed 10 years.

Current Projects

Although the program set up under the Water Supply Act of 1958 is still young, several projects are already in various phases of development. As might be expected, the central-southwest portion of the country has a great number. In this area, as well as other parts of the United States, needs for more water are already acute. Each project presents an opportunity to local interests to plan for future water needs of their community. At the present time, requests have been received from the Corps of Engineers for consultation on 57 projects, 35 of which are in the central-southwest area.

The objective of the program is to assure maximum use of each reservoir site for municipal and industrial water supply storage. The USPHS aids in achieving this objective by virtue of an agreement (Nov. 4, 1958) between the Department of the Army and the Department of Health, Education, and Welfare. USPHS acts as consultant to the Corps of Engineers by evaluating water supply potentials of their projects, such as the storage facilities required for municipal and industrial purposes, and their economic value to the Corps of Engineers. Each project is evaluated on the basis of the demands in the area to be served. Once these demands have been determined, a value must be placed on the water supply storage. This value is used as the basis for judging whether the inclusion of water supply storage is justified in any given project.

Importance of Planning

Domestic and municipal uses are, of course, the most important uses of water. Industrial use follows closely. Because the number of potential dam sites in the United States is limited, the danger of incomplete use of these sites becomes evident. Once built, a dam becomes a part of the geology of the country, with permanent effects. Thus, the inclusion (or exclusion) of water storage at any particular site is a "now-or-never" proposition. Steps must be taken now to insure that each project or proposed reservoir is carefully studied with regard to the advisability of including storage for future water supply needs.

By what means can this objective be attained? The first and most important step is the development of a long-range plan for 50 years or more ahead, covering the municipal and industrial needs for the period. Consideration must be given to population increases, which can only be roughly estimated today, and to the development of new industries. Although USPHS is charged with the responsibility of gathering and presenting the data that determine the inclusion of water supply storage in dam and reservoir projects, it is keenly aware of the sovereignty of the states in water allocations and pollution control. The responsibility for achieving the objectives set forth in the Water Supply Act of 1958 extends to state and local agencies. Indeed, the solutions to some of the problems can be developed only at the local level.

Although water supply planning must be carried out so as to preclude underdevelopment of resources, undue enthusiasm in projecting needs may make project development costs too high to be feasible. Costs must be carefully considered, for, according to the Water Supply Act of 1958, repayment to the federal government by state or local interests for costs apportioned to water supply must be assured "before construction or modification of any project including water supply provisions is initiated." Little could be gained by setting up storage for large future needs unless local agencies, or the states, are willing to assume the costs of facilities to deliver it to the users.

Pollution Abatement

Pollution control is another factor which requires careful consideration. The wastes produced by an expanded population and economy must not be permitted to overload the streams and rivers into which they are discharged. Upstream consumers should not ignore the rights of those downstream to receive water of reasonably high quality.

Although waste treatment has progressed enough that the oxygen demand of organic wastes, such as sewage, can now be reduced by 85–90 per cent, such a reduction is not always sufficient. The remaining demand and other polluting elements may be enough to destroy the usefulness of the receiving stream for many miles. In many instances, the quantity of water used will be dictated by the amount of waste —even treated waste—the receiving stream can assimilate.

The necessity for providing dilution of even highly treated wastes to prevent impairment of water quality to downstream users can present quite a troublesome problem. It cannot be denied that industrial and municipal expansion can be affected or, indeed, limited by the waste load capacity of a local stream. As the law stands, payment for water used for dilution must be made immediately; the problem is to find someone willing to do so. There is some question whether to hold those liable who use the water first and release the wastes or those downstream consumers who want water of an acceptable quality. Such questions must be answered before the full benefit is to be derived from the nation's water resources.

Financial Considerations

Various aspects of the water supply program have presented problems and most of the problems are financial. One, for example, is the determination of the potential money value of the water for which storage space is included in multipurpose projects. The answer, of course, depends on many factors, such as the ultimate use of the water—that is, whether it will be used for municipal or industrial customers —or the availability of other usable water in the given area. Certainly the value of stored water is less in water-rich areas where the supply exceeds the demand. Until a comprehensive system of evaluation is developed, the principle of the "most likely alternative source" must be used. In this method, the cost of water from a source that would likely be used in the absence of the project is taken to be the value of the water from the project. This principle is far from perfect, but it is the best one thus far devised.

Advance planning and provision for future needs bring the problems of having to pay for water supply storage before the need for it has fully developed. Funds may not be available, at the outset, to pay for water supply storage provided for projected needs. This problem is partially solved by the 10-year interest-free period provided by the 1958 act and by the provision that repayment need not precede actual use of the water. If a local community feels that it will be unable to pick up the necessary cost at the end of 10 years, some larger agency must assume the obligation; or a reservoir site may be only partially developed. Limitations placed on the borrowing power of municipalities and prohibitions against expenditure until an actual benefit can be gained could hamper progress.

Water Districts

One possible solution to the problems of planning and financing might be the creation of more water districts or similar authorities. Because the development of water demands is relatively unpredictable on a short-range basis, such agencies should be provided with reasonably flexible powers. They should be set up to run at least 50

years, with provisions for extensions beyond that limit. Of course, they need the practical financing ability to shoulder responsibilities exceeding the capabilities of villages, cities, or counties. The water districts should also be capable of extending the areas they will serve beyond the original boundaries if need be. If such procedures are hampered by the lack of adequate enabling legislation, now is the time to encourage necessary revisions.

Conclusion

The Water Supply Act of 1958 provides an opportunity to face the challenge of providing water for tomorrow at a price that may never again be as reasonable. What is done now will shape the economic future of vast areas of the United States for generations. The Water Supply Act of 1958 presents both an opportunity and a challenge that the water supply profession dares not fail to meet.

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Removal of Radionuclides From the Pasco Supply by Conventional Treatment

Robert L. Junkins

THE municipal water system of Pasco, Wash., is typical in design of many systems for similar populations and climates. The system is quite unique, however, in that the Columbia River water being treated contains radioactive materials. The presence of radionuclides leads to some interesting considerations not usually found in public water systems. Of these, the radiation exposure of those drinking the water and the reduction of this exposure by removal of nuclides at the treatment plant are of particular interest.

The nuclides considered here come from the Hanford Atomic Products Operation (HAPO), which is one of the nation's major plutonium production facilities. The plant, located in southeastern Washington, has been operated since 1946 by the General Electric Co. under contract to the AEC. The reservation and other relevant locations are shown in Fig. 1.

The Pasco treatment facilities are approximately 39 mi downstream from the nearest reactor. Richland, although farther upstream, obtains its raw water from the Yakima River and is therefore not of interest. Kennewick obtains water from the Columbia River through a collector system which would be more difficult to evaluate in terms of water plant efficiency. A small farming community, Ringold, utilizes Columbia River water for irrigation only. For these reasons, Pasco was chosen to illustrate the distribution of radionuclides in a municipal water system and the radiation exposure to those who drink treated or untreated river water.

HAPO System

Eight production reactors are located in the northern portion of the plant reservation. These reactors use treated Columbia River water as coolant on a once-through basis and the effluent returns to the river. The reactor cooling water treatment plants (Fig. 2) differ little in principle from the Pasco city plant. The main difference is in the addition of chemicals to control corrosion of the aluminum process tubes and fuel element cladding. The treated water passes through the reactors, removing the heat of the fission process. Before returning to the river, the water is temporarily retained in basins which provide time for the decay of shorter-lived radionuclides. These basins also make it possible to divert substandard effluents to alternative disposal facili-

ties. The water is returned to the river as shown schematically in Fig. 3. Pipes at the bottom of the river carry the reactor effluent water into the main channel of the river and provide prompt mixing and dilution.

Public Exposure

The cooling water returning to the river with its load of radionuclides is an important source of public exposure to radiation, and the only path of bodily intake to be considered here is through the drinking water. There are other sources of radiation exposure, such as produce containing radio-
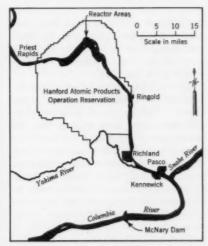
Fig. 1. Hanford Atomic Products Operation and Environs

Of the cities located between the reservation and McNary Dam, Pasco and Kennewick are the only ones using Columbia River water for potable supply. The Kennewick system utilizes radial collector wells and is, therefore, difficult to evaluate in terms of its efficiency in removing radionuclides.
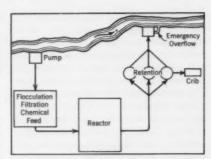
Fig. 2. Schematic Flow Diagram of Typical Reactor Station

The retention basins allow the shorter-lived nuclides to decay, thus reducing the total activity reaching the river. Overly contaminated wastes are diverted to the crib.

nuclides as a result of the use of Columbia River water for irrigation, but complete coverage of all of them is beyond the scope of this report.

As the water passes through the reactors, some of the minerals become radioactive, primarily because of neutron capture reactions. Mn^{56} is one nuclide formed in the reactors by direct neutron addition. Some of the stable Mn^{55} present in the entering coolant adds a neutron to form radioactive Mn^{56}, which has a half-life of approximately 2.5 hr.

Analysis of the quantities of radionuclides by material balance techniques shows that many of the minerals remain in the reactors longer than the flow rate of coolant would lead one to suspect. Film formation in the process tubes and on fuel cladding accounts for the longer exposure of minerals to the neutron flux.

In addition to the neutron reactions mentioned, some radionuclides are produced as a direct result of the fission process. The Columbia River con-

tains trace quantities (approximately 1 ppb) of natural uranium. Not all of the uranium is removed in the cooling-water treatment process and, as a result, some uranium passes through the reactors. Fission products result from this process and are present in measurable quantities. Np^{239} is produced by neutron capture (and beta emission) by the U^{238} present in the cooling water. The same radionuclides are produced in the uranium fuel elements and, occasionally, small quantities of fission products are lost through the failure of fuel element cladding.

Radionuclide Composition

More than 60 radionuclides have been measured in HAPO reactor effluent water, and the concentrations of 27 of them are measured routinely. The group shown in Fig. 4 accounts for more than 95 per cent of the total concentration at the indicated decay times.

In Fig. 4, the radionuclides are arranged from left to right in order of increasing half-life. It is apparent that the relative abundance of the longer-lived emitters increases markedly during the time interval of interest. It should be noted that Mn^{56} is the most abundant radionuclide in reactor cool-

Fig. 4. Isotopic Composition of Reactor Effluent 4 hr After Irradiation and of Columbia River Water at Pasco

Open bars represent percentage activity of effluent 4 hr after irradiation; solid bars represent percentage activity of river water 24 hr after irradiation.

ing effluent 4 hr after leaving a reactor. After 24 hr, however, the Mn^{56} has decayed to a relatively insignificant quantity as compared to other longer-lived nuclides.

The relative abundance of radionuclides is illustrated at two decay times. The choice of 4 hr after irradiation was arbitrary; it has the advantage of excluding radionuclides with half-lives of a few minutes or less. These would be decreased to insignificant levels long before reaching Pasco. It takes approximately 24 hr after irradiation for the waste cooling water to reach Pasco at the nominal river flow rate.

The decay characteristic of the gross mixture of radionuclides during this period is shown graphically in Fig. 5. The experimentally determined curve is the approximate sum of the exponential decay curves characteristic of the individual radionuclides listed previously. It should be noted that the gross content has decreased to a few per cent of the original level by radioactive decay alone. Actually, there is further reduction because of actions in

Fig. 3. Disposal of Reactor Effluent into the Columbia River

By release of the effluent through pipe ends well out in the river, prompt dilution of the radionuclides is obtained.

the river, such as silting and accumulation by aquatic organisms.

Operation of Pasco Water System

The city water plant at Pasco pumps water from the river. Chemicals are added in the treatment plant to form a flocculant precipitate of aluminum hydroxide which removes sediments and particulate matter by agglomeration and settling. The initial addition of chlorine also takes place at the point of entry of the waters which is a mixing chamber and sand trap. The quantities and, to some extent, the kinds of chemicals added are determined by conventional analyses of the raw river water. The turbidity and color of the water are the criteria used to determine the amount of floc required and the need for any coagulating aids such as activated silica. Bacteriologic analyses are utilized to determine the need for chlorine. After addition of the chemicals to the water and the mixing step, the water passes through large sedimentation basins. The floc settles to the bottom of these basins, carrying silt with it. Skimmers remove water from the basins over notched weirs and carry the water to filter beds. These filters provide the final cleanup of solids from the water. The filters are backwashed periodically, and the waste water with its burden of backwash solids is returned to the river. The frequency of backwashing depends on turbidity of the river, the amount of floc used and the water use rate. During the fall and winter months, filters are backwashed as infrequently as once in 100 hr. In spring and summer, backwashing is done every few hours.

The filtered water is collected in a clear well, where additional analyses are performed and the final chlorine adjustment is made as the water is pumped from the clear well to reservoirs. Separate storage facilities are provided in the business district and in the residential areas. The combined capacity of the system, including clear well storage, is slightly greater than 6 mil gal. As the treated water is pumped toward the reservoirs, feeder lines remove water from the mains to supply customers. At times, particularly during periods of high water use, water from the reservoirs is fed back into the system. The time from treatment to actual consumption of water is, therefore, quite variable; it is not
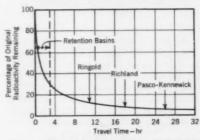
Fig. 5. Reduction in Activity of Reactor Effluent With Time

Data are for a nominal flow rate of 85,000 cfs.

meaningful to state an average transit time figure. The time involved is not of great importance, however, for, as can be seen from Fig. 5, the gross decay curve is rather flat at 24 hr or more after the water leaves the reactor. The remaining radionuclides are fairly long-lived, so that in a few hours more or less the composition would change only slightly.

Sewage Treatment

The city sewage plant receives an average of slightly less than 1 mgd. Because of a storm sewer, this flow increases markedly on the infrequent rainy days. Sewage treatment includes
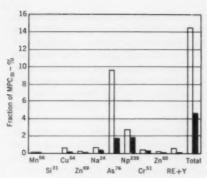
Fig. 6. Comparison of Percentages of MPC$_{GI}$ Present in Raw Columbia River Water and Treated Pasco Water

Solid bars represent average percentages of the MPC$_{GI}$ present in raw river water; open bars represent the same percentages for treated Pasco water. The symbol RE+Y (second pair of bars from the right) means rare earths plus yttrium. The MPC$_{GI}$ values used here are from the National Bureau of Standards Handbook 69, for persons in the neighborhood of a controlled area.

separation of solids and treatment of the liquid before it is returned to the river. The liquid is returned by a submerged pipeline extending to the main channel of the river. The solids pass through digesters. The digestion process requires approximately 1.5 months, during which time the short-lived nuclides are greatly reduced by radioactive decay. Final drying of the solids is accomplished in open basins, after which the material is ground and sold as a soil conditioner. The quantity of radionuclides remaining in the solids is small enough not to be of concern even if the solids are used to fertilize vegetable gardens.

Sampling System

Through the courtesy of the Pasco city officials, HAPO has long main-tained a sampling system at the Pasco water plant, and, at the river pump house, water is continuously sampled. Some samples are passed through ion exchangers to concentrate certain ones of the long-lived nuclides and thus facilitate analysis; other samples are collected and taken to the laboratory for analysis. For short-lived nuclides, however, it is necessary to collect "grab" samples, as integrated samples collected over a long time would result in substantial losses of many of the radionuclides through radioactive decay.

In addition to the raw water at the river pump house, the finished water is also sampled and analyzed. Samples of filter backwash solids are collected and analyzed, as are samples of the accumulated solids in the sedimentation basins, which are drained 2–6 times a year to remove the solids.

Results of the analytic program for a recent year are summarized in Fig. 6. These results do not include all of the nuclides, but the radiation exposure from this group accounts for over 95 per cent of the calculated radiation exposure to the gastrointestinal tracts of the public as a result of drinking
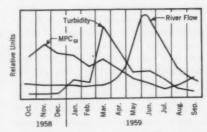
Fig. 7. Seasonal Variations in Turbidity, Flow Rate, and Fraction of the MPC$_{GI}$ Present for the Columbia River at Pasco

As can be seen, peak river flow results in greater dilution of radionuclides.

untreated and treated river water. The radiation exposure of other organs is much less than the values indicated for the gastrointestinal tract. Estimated exposure for bone tissue is about one-tenth that shown for the gastrointestinal tract, for example.

Utilization of maximum permissible concentrations (MPC) as criteria greatly alters the relative importance of the individual radionuclides. For example, in the raw water at Pasco, As^{76} accounts for about two-thirds of the total radiation exposure to the gastrointestinal tract (Fig. 6), although it is much less abundant than Cr^{51} (Fig. 4).

Efficiency of Water Plant

An overall idea of the average efficiency of the Pasco water plant in removing individual radionuclides can be obtained from Fig. 6. The efficiency of the plant in this respect is certainly not constant, however; it varies considerably with seasonal and other influences.

The total activity varies markedly throughout the year. During the peak river flow period, there is, of course, greater dilution of the radionuclides. The seasonal variation of the percentages of the gastrointestinal MPC (MPC_{GI}) is indicated in Fig. 7, which covers the same time interval as the previous illustrations.

The demand for water by consumers varies sharply with the season, mostly because of watering of lawns and gardens during the summer months. Pasco is located in an arid region which receives only about 7 in. of rainfall annually. Figure 10 illustrates these seasonal trends. Note that the flow of sewage remains fairly constant throughout the year, although the use of water increases very sharply during the summer. The sharp peaks in the

sewage flow curve represent infrequent rains.

The change with time in the efficiency of the water plant in removing As^{76} is also illustrated in Fig. 8. During high water usage, the high throughput at the water plant results in less efficient cleanup of the As^{76}. As pointed out above, however, this seasonal trend corresponds to high river flow, which produces a greater dilution of the reactor cooling water and results in lower radiation levels in the drinking water during this period than in the winter months, when the water treatment plant removes a larger fraction of the As^{76}. Other nuclides
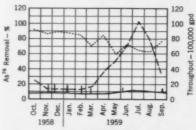
Fig. 8. Seasonal Variations in Water and Sewage Plant Throughputs and Depletion of As^{76} by Water Treatment

The solid, dashed, and dotted curves represent sewage throughput, water throughput, and As^{76} depletion, respectively.

are also removed by processes in the treatment plant. As^{76} was chosen for this illustration primarily because of its importance in terms of permissible limits.

During the winter months, when the flow through the system is reduced (Fig. 8), the water treatment plant discontinues night operation. The reservoir system provides water for consumers' needs during the hours when the water treatment plant is shut

down. Under these conditions, the process time—that is, the time from the river pump house to the clear well pumps—approaches one half-life for As^{76} (26.7 hr). Thus, at such times, an As^{76} reduction of about 50 per cent would be expected from radioactive decay alone.

Waste Solids

Analytic data from the samples of solids from filter backwash and the sedimentation basins are not appropriate for relevant illustrations. As would be expected, the long-lived nuclides predominate, Cr^{51} and Zn^{65} being most abundant. Shorter-lived nuclides decay to low levels while the solids are accumulating, particularly in the sedimentation basins, which require only infrequent cleaning. These solid wastes are returned to the Columbia River.

Cr^{51} and Zn^{65} are the most abundant radionuclides found in the solids removed from the secondary digester of the sewage plant. In the dried solids ready for use as fertilizer, the Zn^{65} concentration (which is about 5×10^{-5} $\mu c/g$) exceeds that of Cr^{51} by an order of magnitude. This relationship indicates the predominance of the longer-lived Zn^{65} at long decay time. Other nuclides present in the finished water are also present in measurable quantities in the liquid wastes leaving the sewage treatment plant. These radionuclides are relatively short-lived, however, and are not measurable in the solid material as it leaves the plant.

Conclusion

The facilities at Pasco were not designed and are not operated with deliberate effort to remove radionuclides from water and sewage. The plants are effective in this regard, however,

and no special precautions are needed because of radionuclides in the water and sewage.

The Hanford Atomic Products Operation has for many years monitored the radionuclides in the city water system and river as a part of an overall environmental monitoring program. As is the case with other portions of the program, the information obtained has provided valuable guidance in the control of Hanford practices as well as assurance that the radiologic quality of Pasco water is well within recognized limits.

Primary emphasis is on adequate control of concentration of radionuclides in the untreated river water which originates at the Hanford reactor. These practices are based on the recognition that there may be some use of untreated river water for drinking water supply. The municipal water plant removes a large fraction of the radionuclides, substantially reducing the amount of radiation exposure that would be received from drinking raw river water.

A comprehensive review of Hanford waste disposal practices and the environmental monitoring program, as well as an interpretation of the results in terms of the several paths of exposure, was presented in testimony before the Joint Committee on Atomic Energy. Hearings on industrial radioactive-waste disposal were held in early 1959.

Acknowledgment

Significance of Radioactivity in Water Supply and Treatment

————————Herbert A. Bevis————————

A contribution to the Journal by Herbert A. Bevis, Chief, Ionizing-Radiation Branch, Div. of Occupational Health, State Dept. of Health, Austin, Tex.

DURING the 63 years since Henri Becquerel discovered natural radiation emanating from uranium, the phenomenon of radioactivity has grown from a laboratory curiosity to one of the most rapidly expanding industries in the world today. The real birth of the Atomic Age came nearly 50 years after original discovery, for it was not until 1942 that Enrico Fermi and his colleagues sustained the first controlled chain reaction in an atomic pile at Stagg Field, Chicago.

It is unfortunate that the first application of this new source of energy was in the form of a devastating weapon, for, to the majority of the public, the atomic clouds over Hiroshima and Nagasaki were the first heralds of the arrival of the Atomic Age.

Much has been said and written concerning the philosophical problems attending the use or misuse of atomic energy, but one thing is certain: the Atomic Age is here, and man must adjust to it, learning to live and work in his new environment.

Radioactivity and Health

Of what importance is nuclear radiation to the water supply field? This question can be answered very simply: Ionizing radiation is harmful to man. Radiation is a form of energy, and if this energy is absorbed by a tissue, injury can occur.

Everyone knows what happens when high-energy projectiles, such as bullets, strike living tissue. Cells are torn apart or injured along the path of the missile until it loses its energy and comes to rest.

Radiation produces a similar effect, but the radiated particles are so minute that they do not dislocate masses as large as cells. Instead, they react with the individual atoms and molecules that make up tissue. Along their paths, the nuclear particles knock electrons out of their orbits and these fly off to cause further damage. The process of ejecting electrons from their atomic orbits is known as ionization. The number of ionizations caused by a radiation per unit of path length is known as its specific ionization. Specific ionization is the major factor in judging the relative biologic effect of various types of nuclear radiation.

To date approximately 30 different nuclear particles have been identified. Of these, only three are of interest to water supply personnel: alpha particles, beta particles, and gamma rays.

Alpha particles are actually identical to energetic helium nuclei. They are relatively large in mass and possess a positive electric charge of two. Be-

cause of their size and charge, alpha particles have a high specific ionization—that is, they dissipate their energy rapidly over a short range.

Beta particles are high-velocity electrons. They are considerably lighter in weight than alpha particles and carry a minus charge of one. They have a moderate specific ionization and penetrability.

Unlike the particulate alpha and beta particles, gamma rays are electromagnetic waves similar to light and radio waves. This wave nature results in a very low specific ionization, but makes gamma rays very penetrating.

Biologic Effects

In discussing biologic effects of radiation, it is convenient to separate the sources of radiation into two classifications: those sources located outside the body and those located within the body. It is apparent that if the source of radiation is an external one, to cause serious injury the radiation must be able to penetrate the skin and ionize the underlying tissue. Alpha particles have an extremely short range—so short, in fact, that they cannot penetrate the dead outer layers of the skin. Obviously, alpha-emitting materials are not external hazards. Beta particles can penetrate into some of the inner layers of skin and cause burns similar to sunburns, but they are not able to reach deeper, more sensitive tissue. Materials that emit beta particles are not, therefore, considered serious external hazards, except to the eyes. Gamma radiation, on the other hand, is able to penetrate the body tissues and cause injury to the critical organs. Table 1 gives the probable effect of various levels of radiation received over a short period of exposure.

The doses listed in Table 1 are far in excess of any possible exposure for water utility personnel, except under the most catastrophic conditions, such as a nuclear war. In addition to the somatic effects of radiation on the person exposed, there are possible genetic effects at much lower levels. These effects would not show up in the exposed individual, but they could appear in future generations.

It is extremely unlikely that anyone would ingest or otherwise take into his

TABLE 1

Probable Effects of Various Radiation Doses to Human Beings

Dose* roentgens	Probable Effect
0–25	No obvious injury
25–50	Possible blood changes, but no serious injury
50–100	Blood-cell changes, some injury, no disability
100–200	Injury, possible disability
200–400	Injury and disability certain, death possible
400	Fatal to 50 per cent
600 or more	Fatal

* A 1-hr exposure to 1 g of radium at a distance of 1 yd would produce a radiation dose of approximately 1 roentgen.

body enough radioactive material to acquire a radiation exposure of the levels presented in Table 1. The greatest concern with regard to internal sources of radiation is the effect of chronic low-level exposure. The actual biologic effect of any radioactive material taken into the body depends on many variables. Some of the more important of these are discussed below.

Quantity of material absorbed. The specific isotope absorbed and its chemical form determine to a large degree

how much of the activity is absorbed from the gastrointestinal tract and taken up by the blood. If, for example, plutonium, one of the highly radioactive materials used in nuclear weapons, is ingested, only a small fraction is taken up by the body. The bulk is not absorbed.

Distribution of radioactive material in the body. If a small amount of radioactive sodium is swallowed, it will be absorbed and distributed widely throughout the body. A gram of shoulder muscle will contain about as much radioactive sodium as the same amount of lung or spleen. On the other hand, radioiodine is absorbed in a highly selective fashion; approximately seven-eighths of it goes to the thyroid gland. In a 150-lb man, the thyroid weighs approximately 0.7 oz. Because of its tiny size, and its very specific absorption of iodine, a high radiation dose may be produced in that tissue. Certain other materials, such as radiostrontium and radiocalcium, are deposited in the bone and can cause considerable damage to the radiation-sensitive bone marrow, where the body's blood cells are formed.

The half-life of the material. Each radioactive material decays at its own constant rate. In a certain period of time, one-half of the original activity will have decayed, in a second similar period of time one-half of the remainder will have decayed, and so on. The time required for the radioactive substance to decay to one-half of its original level is known as its half-life. A material such as radiofluorine, with a 2-hr half-life, cannot be a serious radiation hazard under normal circumstances, as its radioactivity dissipates so quickly. Strontium-90, however, which has a half-life of more than 25

years, remains potent as long as it stays in the body.

The rate at which the material is excreted from the body. The body is continuously rebuilding, and atoms are constantly being replaced, but the rate of exchange varies with the chemical element. The turnover in sodium, for example, is very rapid. On the other hand, radium, plutonium, and strontium-90 become fixed in the bone tissue and remain there for a long time.

Contamination of Water Supplies

The number of potential sources of radioactive contamination of water supplies is increasing day by day. Prior to 1942, only naturally occurring radioactive materials, such as radium or uranium, afforded an opportunity for radiological contamination of water supplies. Since the appearance of nuclear reactors, uranium mills to supply their fuel, and chemical processing plants to handle their spent fuel rods, the contamination of reactor cooling waters and the general availability of radioisotopes have increased the opportunity for radioactive pollution of water. One has only to look at the increased distribution of radioactive isotopes through the last 12 years to get a picture of the rate at which potential problems from this source are growing.

The AEC began to distribute radioactive isotopes to civilian users in 1946. In the subsequent 10-year period, a total of 198,464 curies (c) of activity were shipped from Oak Ridge, Tenn. During 1957, this total grew to 385,826 c, and, by the end of 1958, 608,672 c had been shipped.

Of course, it should not be assumed that the cited amount of activity ultimately reached or will reach public water supplies. Approximately 90 per

cent of the radioactive material distributed by the AEC was composed of cobalt-60 and iridium-192. These isotopes are used in an encapsulated form and are therefore never likely to be discharged as a liquid waste. It should be pointed out, however, that the remaining 10 per cent of the radioisotopes constitutes a sizable amount of activity and its ultimate disposal requires regulation.

To obtain radionuclides from the AEC, a licensee must be able to show that he is competent to work with the material and that he can meet certain waste disposal criteria. These criteria are set up to prevent the disposal of radioactive wastes from developing into a health hazard. Under present regulations it is permissible for certain small amounts of activity to be disposed of by means of the public sewer systems. In an area where there are a number of isotope users, caution must be exercised to prevent the development of a hazard at the sewage treatment plant through the accumulation and concentration of radioactive wastes.

In addition to the control exercised by the AEC, the state departments of health have regulations which cover handling and disposal of radioactive wastes. The states have been assuming more and more responsibility for the control of radioisotopes. Congress passed legislation in 1959 authorizing the AEC to turn over to the states most of its regulatory activities concerning radioisotopes. The AEC will retain supervision of the more hazardous installations, such as reactors and fuel-processing plants.

From the foregoing it is clear that the nuclear industry is fairly well regulated. There is another source of man-made radioactivity, however, over which there is only very limited control.

This source is nuclear weapons. Even if the current cessation of weapons testing is continued by the United States, there is as yet no assurance that other powers of the world will do likewise.

Without question, the testing of more than 200 nuclear weapons to date has raised the level of radioactivity over the entire surface of the earth, surface waters not excluded. Although this increase is quite measurable, it has not reached a hazardous level, according to most experts. Unfortunately, in most areas no one knows what the background radiation levels were before the atomic bomb, and, therefore, no one can accurately assess the increase. In

TABLE 2

MPC's of Five Radioisotopes

Isotope	Maximum Permissible Concentration	
	$\mu c/ml$	ppm
Uranium-238	4×10^{-4}	1200
Carbon-14	8×10^{-3}	1.8×10^{-3}
Radium-226	1×10^{-7}	1×10^{-7}
Strontium-90	1×10^{-6}	5×10^{-9}
Iodine-131	2×10^{-5}	1.6×10^{-10}

any event, levels of radioactive contamination of water supplies are well below the maximum permissible concentration (MPC) recommended by the National Committee on Radiation Protection and Measurements.[1] The MPC's are quite similar to the USPHS Drinking Water Standard. Whereas, for example, USPHS standards suggest maximum allowable concentrations of such things as lead or arsenic in terms of parts per million, the MPC's for specific radioisotopes are given in microcuries per milliliter. Table 2 lists several radioisotopes giving their MPC values in terms of both units.

It is almost impossible to determine isotope concentrations by ordinary chemical means in the vicinity of many of the MPC levels. Radiological techniques are usually employed. This requires special instrumentation which only a few of the larger water plant laboratories possess today. In a few years such equipment will probably be as common as the laboratory balance is today.

Removal From Water

The removal of radioactive materials from either surface or ground waters starts long before the water reaches the treatment plant. In surface water, the physical process of sedimentation reduces the particulate activity, the extent depending upon turbulence of the stream and size of the particles. In addition, many silts have ion-exchange capabilities and can reduce the level of soluble activity. Biologic processes involving both plants and animals may be effective in reducing the radiochemical concentration. For example, some forms of algae have been known to concentrate radiophosphorus by factors as high as 10^5. Unfortunately, the benefit of these natural processes is only temporary, because the radioactivity is only stored in the stream— not removed. During high flows the bottom sediment is resuspended, and, with the change in seasons, plants and animals die, returning the activity they contained to the stream.

Water percolating down through the earth to an aquifer also undergoes some natural treatment. Filtration affords some removal, but the most effective process probably is ion exchange. A number of minerals have very good ion-exchange properties, especially some of the clays. The actual degree of decontamination attained depends on the chemical and physical geology of the area.

Once the water has reached the treatment plant, its radiologic quality is further improved by conventional treatment processes.[2,3] Removals of mixed radioactive fission products of 45–85 per cent can be expected from coagulation and sedimentation, depending upon the amounts and kinds of coagulating chemicals employed. Subsequent filtration may increase these removals—for example, removals of 46 per cent by coagulation and settling were increased to 70 per cent with the addition of filtration.

With lime–soda ash softening, removal efficiencies may be increased to 90–95 per cent with excess treatment.

Ion-exchange treatment is very efficient in removing radioactive materials. In mixed beds containing both cation and anion resins, removals in excess of 99.99 per cent have been obtained. The efficiency of such units, however, is dependent entirely upon the chemical composition of the water under treatment. In general, removals of radioactive materials parallel hardness removal, although some removal (50–75 per cent) of the radioactive fraction may continue even after the bed's hardness removal capacity has been exhausted.

Some attention has been directed toward the designing of special units to decontaminate water supplies.[4] One of the most promising units developed is one containing beds of steel wool, clay pellets, activated carbon, and cation-anion-exchange resins in series. Water containing a mixture of radioactive materials was passed through this unit and a removal of 99.996 per cent was obtained. Distillation units should also be capable of effecting removals of the nonvolatile materials up to 99.99 per

cent or better. Such units would probably be expensive and quite limited in capacity.

It should be emphasized that treatment of water containing radioactivity does not destroy the activity. It simply removes the contaminated material from the water and deposits it in the sludge or ion-exchange bed. When grossly contaminated waters are treated, special caution may be required in handling the materials in which the activity has been highly concentrated.

Conclusion

In addition to the radiologic materials which may find their way into water supplies, the nuclear-energy program has spawned interest in many materials that were formerly relatively unknown. Rare earths are rare no longer. Beryllium and other less well known metals have found unique applications. Some of the new materials are very toxic. The properties of others have not been determined. It would not be surprising if the everyday vocabulary of analysts in water plant laboratories within the next few years came to include the names of such elements as promethium, cerium, and europium.

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Progress in Chemical Handling and Feeding at Chicago

Oscar Gullans

A paper presented on Mar. 18, 1960, at the Illinois Section Meeting, Chicago, Ill., by Oscar Gullans, Chief Filtration Engr., Water Purif. Div., Chicago, Ill.

SINCE the beginning of its experiments approximately 10 years ago, Chicago's South District Filtration Plant has come a long way in the handling and feeding of liquid chemicals and slurries. In the hope that some of the lessons learned in these 10 years of experimentation may be of interest to others, a few of the more recent developments are described below.

Activated Carbon

From experience with heavy feeds of activated carbon during prolonged periods of heavy pollution in 1959, adequate storage capacity was shown to be a great asset. Experiments have established that activated carbon can be stored in water for a year without appreciable change in its adsorptive capacity. At the South District plant during the winter months, a carbon concentration up to 1.5 lb/gal is used in the slurry, instead of the usual 1 lb/gal. This increases the carbon storage capacity by 50 per cent without additional tanks. This practice created one problem, however. The heavier carbon slurry, which had to be pumped a vertical distance of 70 ft, had a tendency to leave plugs of carbon at the bottom of vertical pipe sections. The practice was adopted of flushing the pump and lines with clear water long enough to clear the pump and lines after each operation. Occasionally a storage tank is loaded with a concentration of more than 1.5 lb/gal. When this occurs, the contents of tank are thoroughly mixed, a portion of the carbon is pumped to the feeding tanks, and the balance is diluted to the proper concentration. The use of the carbon slurry has not only proved to be the most economical method of handling this material; it has also been invaluable when heavy feed rates were required. At present, the carbon-feeding equipment has a top feeding capability of 15,000 lb/hr.

Liquid Alum

In the liquid alum system, which has not been materially changed in 3 years, the problems of foaming and slime growths encountered 3 years ago have been practically eliminated by discontinuing the practice of diluting the liquid in the storage tanks. By arrangement with the manufacturer, the liquid is now received at a concentration that does not produce crystallization on cooling. Any dilution of liquid alum seems to upset the chemical equilibrium of its normal constituents. By using the undiluted material of the proper concentration, it has been possible to eliminate practically all feeding problems.

Ferrous Sulfate

The handling and feeding of liquid ferrous sulfate has been somewhat im-

proved in the past 3 years. Additional feeding tanks have been added to increase storage capacity. Because the material is a byproduct of pickling operations at steel and wire plants, a system of decanting the liquid from the receiving tank to the storage tanks has been adopted. Occasionally the liquid contains fine particles of ferric oxide, which settle to the bottom of the receiving tank and can be drained off periodically. The iron concentration varies with the source of the liquid, and it is occasionally necessary to dilute the contents of the receiving tank before decanting in order to prevent crystalization of ferrous sulfate. The dilution with water seems to have no deleterious effect on the characteristics of the liquid material. It has also been found advantageous to filter the material through a coarse fabric filter as it is being unloaded from the delivery tank truck. This procedure avoids the possibility of allowing foreign matter to drop into the receiving tank.

Fluoride

Hydrofluosilicic acid has been used for fluoridation at the South District plant for approximately 4 years. The only problem thus far encountered has been in the maintenance of the feed pumps. Because the liquid is corrosive, some changes have been made in the style and type of pump used to feed the concentrated liquid directly to the water in a closed system. The feed pumps have been converted from the straight-plunger type to a diaphragm type, which appears to be more satisfactory. After a few weeks' operation the plunger packing on the original pumps required frequent adjustment or repacking to prevent leakage of acid. The diaphragm type now being used is a piston-type, positive-displacement

pump with an activated diaphragm. It is more reliable and requires less attention, and the only leakage through pump glands is noncorrosive hydraulic oil.

Sight-flow ball check devices, which were fabricated from plastic in the plant's own shop, have been installed in the discharge lines. On several occasions they have indicated clogging of the feed lines by air or foreign material, and immediate corrective action has been taken. Some difficulty has been experienced with the bubbler-type level gages on the storage and feeding tanks. The bubbler lines have clogged and have had to be blown out with air. The air supply to these lines has caused the pressure-reducing valves and flow controller to become so fouled with oil and moisture that they have had to be removed for cleaning. A float-type elevation gage is currently being tried out in each large feed tank. This gage indicates tank elevations to within 0.01 ft, and it is believed that it will be more reliable and require less attention. It is also planned to install two small feed tanks with a capacity of only 200 gal per foot of depth, in order to be able to check pump feed rates more accurately. Each of these tanks will have about one-tenth of the capacity of the present feed tanks, and will insure close control of the feed rate. There is no sign of corrosion in the rubber-lined steel tanks or polyvinyl chloride pipe since their installation 4 years ago.

Experiments have also been made with a gear-type feed pump as a measuring device for hydrofluosilicic acid. The first such pump, which was made of a high grade of stainless steel, began to show wear within a few days. A second pump was made of another alloy and also showed signs of wear after 2 weeks. Further experiments in the

design of this pump are being conducted by the manufacturer to develop a unit that can handle the job.

Sodium Silicofluoride

Because hydrofluosilicic acid may not be available at some future date, a series of experiments has been run during the past 2 years for the handling and feeding of sodium silicofluoride. These experiments were conducted both on a laboratory and on a pilot plant scale.

The first experiment tried was to dissolve the fine granular material in a saturator or percolation tank of the solids-contact type. The first tests were run in a clear plastic tube 6-in. in diameter and 10 ft high, with a few inches of filter gravel in the bottom to act as a water distribution grid. The tube was usually filled with 6–8 ft of material. Dissolving tests were made on the pure sodium silicofluoride and on a low-grade fluoride material that showed possibilities of considerable saving in fluoride costs. Because it takes 30–60 gal of water, depending on the temperature, to dissolve 1 lb of the sodium silicofluoride, a diluted alum solution was used to try to increase the solubility with the same equipment. A 5 per cent alum solution more than doubled the solubility of the fluoride.

A larger steel tank with a diameter of 3.4 ft and a height of 10 ft was then used. A series of dissolving tests was made using the two types of material. The fluorides could be dissolved in approximately the same ratio as was indicated in the laboratory tests. The back-flushing rates required to supply enough fluoride for one-third of the plant requirements meant that a velocity in excess of 0.2 fpm was necessary. Under these conditions, it was hard to

prevent carryover of the finer crystalline material, and there was a tendency for channels to be dissolved through the lower portion of the bed. This problem could be met by running the discharge from the percolation column into a larger settling tank, from which the clear fluoride solution could be drawn and the slurry at the bottom pumped back to the percolator. As a result of these tests an easier method was sought.

With the same type of percolation columns and equipment, large numbers of tests were run on pellets of sodium silicofluoride, in the hope that this would prevent the carryover of fine crystals. Both fluoride manufacturers supplied us with generous quantities of sodium silicofluoride pellets made from material finer than 200 mesh. The fluoride worked very successfully both with and without a 5 per cent alum solution as the dissolving agent. The lower-grade fluoride was difficult to compress into pellets, and the carryover problem was still present when the pellets began to disintegrate in the percolation column. Solubility rates were only slightly less than with the fine material. It is believed that the pelletized material would work very well in a percolation column, but its use would necessitate the added cost of pelletizing.

Experiments are now being continued on the possibility of adding the fine fluoride powder as a slurry. Laboratory experiments have indicated that such a slurry dissolves quite rapidly in the mixing basin (within 2–3 min), with little or no settling of the undissolved material. One peculiar feature indicated by the tests is that a slurry concentration of 60–70 per cent solids provides the best working range. Using pure sodium silicofluoride of

−200 mesh, or the lower-grade fluoride, which is −300 mesh, a 70 per cent concentration could be in tanks for several days without appreciable settling. Mixing equipment and small pumps are presently being tested for handling such a slurry. The ultimate goal is to receive the dry sodium silico-fluoride in bulk in hopper cars or trucks, transfer it to a slurry tank for storage, and then pump it to the feeding equipment. Using a 60–70 per cent concentration, the feed pump would be sized at 0.2–2.0 gpm in order to provide a feed rate sufficient for the needs of the entire plant. This proposed setup would require only two medium-sized tanks for slurry mixing and storage, two small feeding tanks, two small transfer pumps, and two small feed pumps equipped with adjustable feed control. Such equipment would reduce handling and maintenance to a minimum, with accompanying lower cost for the basic material.

Lime

The use of pebbled quicklime has posed problems in handling and feeding. In addition to the inherent dust problem there have also been difficulties in the maintenance of slakers and lime application lines as well as in grit disposal. This problem has persisted for a number of years.

To solve the problem of carbonate encrustation of feed lines, they have been replaced with steel troughs from the lime feeder to the point of application. The troughs have removable covers and can be cleaned periodically. Fortunately, the distance from the lime slaker to the point of application is short. In the new Central District Filtration Plant now under construction, the distance from feeders to points of application will be much greater and the lime will have to be pumped instead of using a gravity flow.

Extensive experiments are being conducted on the possible use of hydrated-lime slurry at the South District plant. These experiments have indicated that the best slurry for handling and pumping is one with a 40–45 per cent solids content. The dry hydrated lime was added to a slurry-mixing tank equipped similarly to the ones used for carbon slurry. Several types of pumps have been tried, and it has been possible to feed the 40 per cent lime slurry from the mixing tank through 600 ft of 1-in. plastic hose to the point of application. To date three carloads of hydrated lime have been used in these tests. As the hydrated-lime slurry settles very slowly at this concentration and appears to remain in the fluid state from bottom to top for more than a week without agitation, it would not be necessary to flush the long feed line for short shutdown periods. Carbonate deposition inside the long feed line appears to be negligible. On one or two occasions small slugs of hydrated lime have collected in the lime so that the pump created sufficient pressure to blow off one of the tubing connections. The formation of the lime slugs was attributed to the fact that the feed pump was started before the slurry, which had been standing for some time, could be completely suspended in the tank.

Before the final design of the slurry equipment for the South District plant, further tests will be conducted on types of mixers and pumps. Although the hydrated-lime cost is slightly higher than the cost of pebble quicklime, simplification of handling and feeding equipment and elimination of the problems inherent in the use of quicklime will make the change worth while.

Ammonia

Ammonium sulfate has been used at the South District plant for many years in chlorine-ammonia treatment of the filtered water. Most water plant operators are familiar with the feeding problems connected with this product. In addition, there are the problems of receiving the material in bags, moving it to and from storage, dumping it into the feeding bins, and maintaining the dry-feed equipment. These problems are being solved by the installation of anhydrous-ammonia storage and feeding equipment. A new 12,000-gal anhydrous-ammonia storage tank, from which the ammonia will be drawn as a gas through conventional feed equipment and will be fed as a gas at the point of application, is being installed. This will greatly simplify handling and feeding and will result in a considerable saving in maintenance and material costs.

Tank Linings

Experiments have been conducted on the lining of concrete or steel tanks that hold the liquid chemicals. Five different materials are currently being tested, all but one of which have proved to be satisfactory to date. The exception is a heavy sheet-plastic lining that failed in two ferrous sulfate tanks after only a year, possibly because application methods were poor. The lining continued to leak after several attempts were made to seal the seams, and it finally sagged away from the walls, although the lining itself shows no sign of deterioration. There are two possible explanations, one being that the concrete surface was not completely cured, and the other that the wrong type of adhesive was used to attach the sheet lining to the tank wall.

Two 10,000-gal concrete ferrous sulfate tanks are lined with liquid neoprene, applied by brush, and four 10,000-gal tanks used for liquid ferrous sulfate or aluminum sulfate are lined with epoxy resin coatings. All of these tanks are converted concrete bins originally used for dry-chemical storage. Three of the epoxy resin coatings were applied by plant personnel and one by the manufacturer. Fiberglass was used to build up the coatings to a thickness of 12–20 mils. After 2 years of service, none of the coatings show any effect from liquid-chemical storage. Three more concrete storage tanks are in process of being lined, two with epoxy coatings and one with sheet neoprene.

Conclusion

Interest in developing simplified andling and feeding methods for water treatment chemicals is motivated by construction of the new Central District plant. A large number of problems encountered in the early operation of the South District plant have been eliminated. In addition to reducing the handling costs, the use of liquid chemicals or slurries will greatly reduce the amount of feeding equipment and, therefore, maintenance cost at both plants. This is also reflected in the reduced space requirements in the new structure. At the time when the South District plant was constructed, it had the best handling methods and chemical feeding equipment available. It is hoped that this can be said of the new Central District Filtration Plant.

Use of Sodium Aluminate as a Coagulant at Council Bluffs

—Myron E. Rew—

FOR years, sodium aluminate has been of relatively minor impor-tance for the coagulation of municipal supplies. It was principally used in hot-process softeners, boilers, and for various special coagulation operations. Despite its obvious advantages as a concentrated form of alumina and an alkaline softening agent, sodium alumi-nate was not generally accepted as a primary coagulant for clarification of large municipal supplies. In most in-stances, it could not compete with the acid coagulants on a cost-performance basis.

One grade of sodium aluminate was given a short trial run at the Council Bluffs, Iowa, water plant in 1953, but it did not produce the desired results. Recently, a new form of sodium alu-minate * was used with considerable success. Those interested in water-softening processes may find valuable the results obtained with this alkaline, alumina coagulant at the Council Bluffs plant during 1959, as compared with the results obtained with alum in 1957, the last complete year during which it was used.

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* Nalco 614, manufactured by Nalco Chemical Co., Chicago, Ill.

Treatment Plant

The plant at Council Bluffs consists of preliminary settling basins, followed by two stages of treatment basins and eight 1.5-mgd sand filters. The plant has been in operation continuously since 1952. It is operated as a two-stage, split-treatment, clarification and softening plant. Water is pumped from the Missouri River to the pre-liminary settling basins, where chlorine is applied. The retention time here is 5 hr at a flow rate of 10 mgd.

The settled water then flows by gravity to the treatment basins, where approximately 75 per cent of the set-tled water enters the first-stage flash mixer; the other 25 per cent enters the second-stage flash mixer. Sufficient lime is added to the water in the first-stage mixer to remove all possible carbonate hardness. A caustic alka-linity of approximately 40–50 ppm is maintained here to remove the mag-nesium hardness as the hydrate. Sludge, composed of magnesium hy-droxide and calcium carbonate, is re-turned from the first-stage clarifier basins to the first-stage mixer. This sludge is about 1 per cent by volume of the total throughput of the plant.

After the addition of lime and returned sludge, the water flows to the first-stage flocculator basins, where the softening reactions are completed and the floc is developed. The retention time in these floc basins is approximately 1.25 hr, at a flow rate of 10 mgd. After the floc is developed, the precipitate settles out in the clarifier basins. The large, heavy magnesium hydroxide floc developed with the excess lime drops out rapidly, carrying

the preliminary basins. The coagulant is added at this point. Soda ash is also added here if removal of noncarbonate hardness is desired. Sludge is returned to the second-stage flash mixer from the second-stage clarifier basins. This sludge amounts to approximately 1 per cent by volume of the total throughput of the plant. The sludge is composed of calcium carbonate and aluminum hydroxide. The solids content of the sludge is approxi-
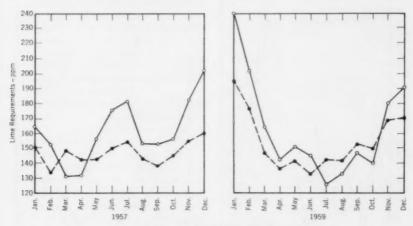
Fig. 1. Lime Requirements at Council Bluffs

The solid curve shows the amounts of lime actually applied; the dashed curve, the amounts theoretically required.

with it the calcium carbonate formed in this stage and leaving very little turbidity in the water. The retention time in these basins is approximately 3.25 hr, at a flow rate of 10 mgd.

The water then flows through the first recarbonation chamber—where carbon dioxide is added if necessary to supplement the split treatment—to the second-stage flash mixer basin. Here it is mixed with the remaining 25 per cent of the settled water from

mately 20–30 per cent by volume after 1 hr of settling.

Water from the second-stage mixer flows to the second-stage flocculator basins, where the noncarbonate softening and calcium carbonate stabilization is accomplished and the floc developed. Water flows directly from these flocculator basins to the final clarifier basins, where the calcium carbonate and aluminum hydroxide precipitates settle out and are recirculated

or removed. The retention time is slightly shorter in this second stage because of the slightly larger volume of water.

From the final clarifiers, the water flows through the final recarbonation chamber, where further pH reduction

nesium hydroxide is present, it is necessary to use a coagulant to help flocculate and speed the settling of the calcium carbonate, so that water with excess turbidity will not be delivered to the filters. The most commonly used coagulants, aluminum sulfate and
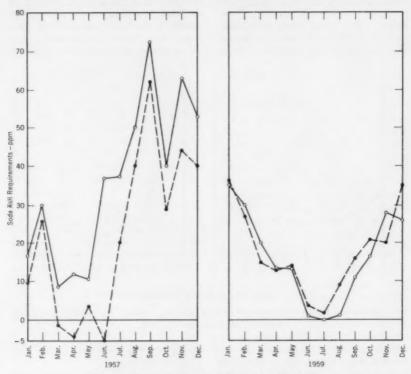
Fig. 2. Soda Ash Requirements at Council Bluffs

The solid curve shows the amounts of soda ash actually applied; the dashed curve, the amounts theoretically required.

is accomplished if necessary. From this chamber, the water flows to the filters.

Coagulation

In the second stage of the softening treatment, when practically no mag-

ferric or ferrous sulfate, contribute hardness to the water and therefore create the need for an additional softening chemical to compensate for the added hardness. For many years, a coagulant that would not add hardness to the water has been sought.

The new polyelectrolyte coagulant aids that have been developed in recent years are effective and represent a real advance in water treatment. Most of these new aids, however, are added as supplemental treatment, but, though they may reduce the coagulant requirements and improve the floc, the primary coagulant used will still add some hardness to the water.

Results of Trial Use

At the Council Bluffs plant, and at many other plants designed to treat 5–15 mgd water, the operators handle the routine tests to control the treatment process and make adjustments accordingly. The use of multiple coagulants or coagulants and coagulant aids might be confusing to them. Although the operators at Council Bluffs know the fundamentals of water treatment, it is desirable to keep the treatment as basic and clearcut as possible.

When it was discovered that the new sodium aluminate material was used successfully at Ottumwa and Dubuque, Iowa, a study was made of the results. It was decided to give the new coagulant a trial at the Council Bluffs plant.

At the second-stage mixer, the split treatment—in which is utilized the 25 per cent of the water that bypassed the first stage—is used to neutralize the excess caustic alkalinity and reduce the high pH resulting from the lime treatment in the first stage. The pH is reduced from a range of 10.3–11.0 to a range of 9.3–9.8. If necessary, carbon dioxide gas is used to effect some of this reduction. The coagulant is applied at this mixer along with soda ash, chlorine, and sludge returned from the second-stage clarifiers. Calcium carbonate stabilization is thus accomplished at the second-stage mixer. Fine particles of

calcium carbonate remain in suspension and carry over onto the filters, unless a coagulant is used in the clarifiers to form a floc and remove the particles. No attempt, however, is made to keep turbidities from the second stage extremely low. Instead, water with 5–10 ppm turbidity is allowed to flow on the filters.

During the first month of treatment (August 1958) with this new coagu-

TABLE 1

Costs and Materials Used With Alum and With Sodium Aluminate

Item	1957	1959
Water treated—*bil gal*	1.740	1.860
Alum		
Used—*lb*	310,000	
Applied—*ppm*	21.4	
Cost—*$/cwt*	2.75	
Total cost—*$*	8,500	
Cost—*$/mil gal*	4.90	
Sodium aluminate		
Used—*lb*		80,000
Applied—*ppm*		5.1
Cost—*$/cwt*		10.11
Total cost—*$*		8,100
Cost—*$/mil gal*		4.35
Soda ash		
Used—*lb*	510,000	220,000
Applied—*ppm*	35	14
Cost—*$/cwt*	2.25	2.25
Total cost—*$*	11,500	4,950
Cost—*$/mil gal*	6.60	2.65
Noncarbonate hardness reduced—*ppm*	23	17.5
Cost of noncarbonate hardness reduced—*$/ppm*	0.29	0.15
Lime		
Used—*lb*	2,370,000	2,470,000
Applied—*ppm*	164	160
Cost—*$/cwt*	0.98	0.98
Total cost—*$*	23,200	24,200
Cost—*$/mil gal*	13.35	13.00
Alkalinity reduced (as $CaCO_3$)—*ppm*	109	129
Magnesium reduced (as $CaCO_3$)—*ppm*	37	45
Total alkalinity & magnesium reduced—*ppm*	146	174
Cost of alkalinity & magnesium reduced—*$/ppm*	0.091	0.075
Overall Values		
Noncarbonate hardness reduced—*ppm*	23.0	17.5
Alkalinity & magnesium reduced—*ppm*	146.0	174.0
Total hardness reduced—*ppm*	169.0	191.0
Cost of alum—*$/mil gal*	4.90	
Cost of sodium aluminate—*$/mil gal*		4.35
Cost of soda ash—*$/mil gal*	6.60	2.65
Cost of lime—*$/mil gal*	13.35	13.00
Total cost of materials—*$/mil gal*	24.85	20.00
Total cost—*$*	43,240	37,200

lant, the operation proved to be educational as well as successful. It was found that for production of the best floc, the pH had to be 9.3 or greater. With the new material, coagulant requirements were much more sharply defined and quite simple to control. Soda ash requirements were substantially reduced. The results of the first-month's trial justified the continued use of new sodium aluminate for studies of its reaction in cold waters of winter operation and also in the runoff water of spring, which is usually very difficult to treat successfully.

Further Studies

With the sodium aluminate, coagulation and floc formation in cold water (34°–35°F) proved very effective with dosages of 3–4 ppm. Springtime treatment was difficult, but the results were as good as those during runoffs in previous years. Results were considered satisfactory, although dosage requirements were sometimes as high as 8–10 ppm.

Only one difficulty has been encountered so far, though it has not yet become serious. During the last few months, floc formation with sodium aluminate alone has not been as good as it was in the beginning. The reason for the difficulty is not yet known. The problem has been solved, however, with the addition of 5–7 ppm alum along with the sodium aluminate. This seems to trigger a reaction that brings about the formation of a good floc particle with quite satisfactory results, even during seasons of the year when the water causes most difficulty.

The use of a second chemical to trigger the reaction creates a slight inconvenience, but the advantages of the new coagulant offset this. During extended use of the sodium aluminate in conjunction with the softening treatment, a definite reduction in lime requirements for alkalinity and magnesium removal and a reduction in soda ash requirements for noncarbonate hardness removal were achieved (Fig. 1 and 2). Also, the unexpected advantage of better filter bed conditions became clearly apparent after continued operation.

Of some concern is the rapid increase in the price of the sodium aluminate which has taken place since it was put on the market. Current prices were used in comparison studies, and still a very substantial savings is shown (Table 1).

Summary

The use of new sodium aluminate in conjunction with lime–soda-ash softening at the Council Bluffs treatment plant has proved successful and has several advantages over the use of alum:

1. Coagulant requirements are well defined and simple to control.

2. Although the cost of 1 ppm sodium aluminate is equivalent to the cost of 3.68 ppm alum, the smaller dosage requirements of sodium aluminate make it more economical than alum.

3. Lime requirements are smaller for alkali and magnesium reduction.

4. Soda-ash requirements were substantially reduced, resulting in significant cash savings.

5. Filter beds stay in better condition.

The use of this new sodium aluminate will be continued. It is believed that future results will be as successful as those of the past.

Recovery and Reuse of Alum Sludge at Tampa

————J. Moran Roberts and Charles P. Roddy————

A paper presented on May 18, 1960, at the Annual Conference, Bal Harbour, Fla., by J. Moran Roberts, Vice-Pres., Robert & Co. Assocs., Atlanta, Ga., and Charles P. Roddy, Chemist, Tampa Water Dept., Tampa, Fla.

THE production of aluminum sulfate from aluminum hydroxide sludge, or hydrous aluminum oxide sludge, and sulfuric acid is not new. Several water treatment plants throughout the United States have produced alum at their plant sites for many years. These plants use bauxite ore as their source of aluminum oxide. The only difference between the hydroxide and the oxide is the water of hydration in the former. The hydroxide is also used in a commercial process for the production of an iron-free alum.

Probably the earliest attempt to reclaim alum sludge was made by Jewell,[1] who, in 1903, patented a process for water treatment and for reclaiming the coagulant used by reacting the aluminum hydroxide with sulfuric acid. Mathis,[2] in 1923, was issued a patent for basically the same process as Jewell's. In a report for the city of Orlando, Fla.,[3] on the disposition of water plant sludge, Black Laboratories, Gainesville, Fla., in 1951, suggested the use of the alum sludge recovery process utilizing sulfur dioxide from the gas of boiler stacks as a source of sulfuric acid. Tampa, Fla., started investigating the feasibility of the process in 1956. Facilities for acid storage and handling were available at the treatment plant, and concentrated sulfuric acid could be purchased locally, making the process particularly suitable. Laboratory tests indicated that the chemistry and control of the process would be rather simple. The major problem was to determine what method of liquid-solid separation would be most practical for thickening the alum sludge, for it is a difficult material to concentrate. The preliminary work indicated the need for additional testing in a pilot plant.

Tampa Supply

Tampa gets its water supply from the Hillsborough River, a highly variable source that changes in quality considerably throughout the year, depending on the amount of rainfall. During the wet season, when the flow is largely from rainfall and runoff, the organic color of the water increases and the mineral content decreases. During the dry season, when the flow is mostly from spring or ground water, the organic color decreases and the mineral content increases. The quality of the raw water usually falls into three general classifications:

1. Small amount of organic color and large mineral content
2. Large amount of organic color and large mineral content

3. Large amount of organic color and small mineral content.

In all categories, the concentration of inorganic suspended solids is small.

Alum is used as a coagulant; activated silica, as a coagulant aid. Lime is used for stabilization ahead of the filters when the color is present in large amounts. When the water has a high alkalinity and a small color content, lime is used for softening with carbon dioxide stabilization ahead

water analysis and alum dosage are shown in Table 1.

In August 1956, Tampa began using liquid alum, after having used dry alum for almost 30 years. Because laboratory results indicated that a considerable savings in alum could be achieved by the use of sulfuric acid to lower the pH to the optimum coagulation zone for color removal, facilities for acid storage and handling were constructed along with those for liquid alum. Ac-

TABLE 1

Analysis of Raw Water—1958*

Month	Color†	Total Alkalinity (as CaCO₃) ppm	Total Hardness (as CaCO₃) ppm	Noncarbonate Hardness (as CaCO₃) ppm	pH	Alum Dosage ppm
Jan.	60	92	117	25	7.5	69
Feb.	80	64	86	22	7.3	82
Mar.	129	38	49	11	6.8	81
Apr.	130	50	63	13	6.9	94
May	85	82	97	15	7.3	100
Jun.	41	109	130	21	7.4	59
Jul.	110	68	88	20	7.0	102
Aug.	118	63	76	13	6.9	110
Sep.	100	82	98	16	7.1	107
Oct.	74	97	116	19	7.3	95
Nov.	67	98	119	21	7.4	82
Dec.	50	107	126	19	7.6	56
Max.	130	109	130	25	7.6	110
Min.	41	38	49	11	6.8	56
Avg	87	79	97	18	7.2	86

* Average values given.
† Determined by visual comparison with the use of nessler tubes.

of the filters. From the foregoing, it is apparent that water treatment at Tampa entails many different methods, and almost all the various means of water treatment are used at times during the year. Tampa has a very flexible plant to treat a water varying greatly in quality. Because color removal is one of the types of treatment used, very high alum dosages are required. The average values for raw-

tual tests on a plant scale showed a 40–50 per cent reduction in alum dosage for some types of water. The acid was used only when the water had a high alkalinity and large content of organic color. When the alkalinity was low, it was difficult to avoid free mineral acid in the treated water. Table 2 shows adjusted dosages of, and savings obtained with, alum and acid, as compared with alum alone.

Recovery Process

When commercial aluminum sulfate is added to the raw water, it reacts with the natural alkalinity. The following reaction is presumed to take place:

$$Al_2(SO_4)_3 \cdot 14H_2O + 3Ca(HCO_3)_2$$
$$\rightarrow 2Al(OH)_3 + 3CaSO_4 + 14H_2O + 6CO_2.$$

As the raw water has less than ten units of turbidity, the resulting precipitate will consist mostly of aluminum hydroxide, organic color (of vegetable origin), and the equivalent of less than 5 ppm activated silica added as a coagulant aid. For each

for disposal. The river waterfront is fast developing into a prominent residential area, and, no doubt, the removal of sludge from the river will become more important soon.

Aluminum hydroxide is an amphoteric compound and will thus ionize as a base or an acid. The following reaction with the acid forms aluminum sulfate:

$$2Al(OH)_3 + 3H_2SO_4 \rightarrow Al_2(SO_4)_3 + 6H_2O.$$

Approximately 1.9 lb of sulfuric acid is required to react with each pound of aluminum hydroxide. As determined from the combining weights of

TABLE 2

Dosages of, and Savings Obtained With, Alum and Acid, as Compared With Alum Alone

Alum			Acid			Cost of Alum Plus Acid $/mil gal	Savings $/mil gal
Dosage		Cost* $/mil gal	Dosage		Cost† $/mil gal		
ppm	lb/mil gal		ppm	lb/mil gal			
102.60	856	15.41	0	0	0	15.41	0
51.30	428	7.70	29.90	249	2.49	10.19	5.22
42.75	357	6.43	38.47	321	3.21	9.64	5.77
51.30	428	7.70	47.00	392	3.92	11.62	3.79

* At $36 a ton.
† At $20 a ton.

ton of commercial alum applied, approximately 525 lb of aluminum hydroxide sludge is formed, in addition to the absorbed and adsorbed material engulfed in the floc. A total of 3,279 tons of alum was used in 1958. This required the disposal of 861 tons of aluminum hydroxide sludge or 557.4 tons dry weight of aluminum oxide. If most of this sludge is recovered, the disposal problem will be eliminated. Disposal of sludge will probably become a troublesome problem sometime in the near future, because the river, below the intake, is now used

the two compounds, the quantity of each chemical required to produce 1 ton of alum is:

$$Al(OH)_3 = \frac{2 \times 78 \times 2,000}{450} = 693 \text{ lb}$$

$$H_2SO_4 = \frac{3 \times 98 \times 2,000}{450} = 1,307 \text{ lb.}$$

Some of the organic material will be carbonized by the concentrated sulfuric acid while the reaction is taking place.

The sludge can be dried and then reacted with the sulfuric acid. The
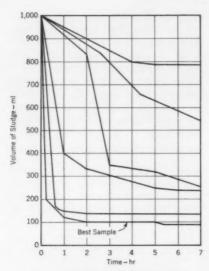
Fig. 1. Settleability Curves for Various Types of Sludge

The best sample contained 0.51 per cent solids by weight and 0.27 per cent aluminum oxide, and was equivalent to a 1.59 per cent commercial alum solution.

oxidation of the organic material is accompanied by its more extensive carbonization. The alum thus produced can be used as a liquid, or it can be crystallized and used in a dry state. Because the dried sludge has enough organic material to support its own combustion—and, therefore, its own destruction—it could be burned, leaving aluminum oxide in a refined form for the production of a more refined alum. The reaction for both dry methods is:

$$Al_2O_3 + 3H_2O + 3H_2SO_4 + 8H_2O \rightarrow Al_2(SO_4)_3 + 14H_2O.$$

In experimental studies, air-dried sludge from a drying bed contained 25–30 per cent aluminum oxide. Oven-dried sludge at 105°C contained

45–50 per cent, which is the same percentage found in some grades of bauxite ore. Therefore, as determined from the combining weights of the two compounds and by use of the 50 per cent value, the amount of dried sludge and acid required to produce 1 ton of commercial alum can be calculated:

$$Al_2O_3 = \frac{102 \times 2,000}{594} = 343 \text{ lb}$$

$$\frac{343 \times 100}{50} = 686 \text{ lb}$$

$$H_2SO_4 = \frac{3 \times 98 \times 2,000}{594} = 990 \text{ lb}$$

$$\frac{990 \times 100}{93} = 1,065 \text{ lb.}$$

Because the wet method of recovery requires only a small amount of equipment for a continuous process, most of the research was carried out with this type of system in mind.

Description of Research

The first phase of the investigation was made in the laboratory on small batch samples of sludge. Because the concentration of raw sludge is 0.05–

Fig. 2. Sludge-Thickening Pilot Plants

Two thickening units—a flat-bottom rectangular tank and a cylindrical tank—were used to determine the maximum thickening that could be obtained at various flow-through periods.

0.20 of 1 per cent by weight at the sludge discharge from two 15-mgd suspended-solids-contact process basins, raw sludge was used as a source of all samples. The physical difference in the type and structure of the sludge varied considerably, as shown by the settleability curves in Fig. 1. Some types consolidated within minutes to a rather dense sludge; others took several hours to reach the same degree of compression. In part, these differences were the result of the type of floc formed during the coagulation of this highly variable surface supply. The maximum thickening obtained with 1,000-ml samples in a graduated

claimed alum necessary for good coagulation. Adjustment of the pH was all that was necessary to insure complete conversion of the aluminum hydroxide to aluminum sulfate. This eliminated some of the time-consuming gravimetric procedures in the aluminum oxide determinations. The pH range was between 1.5 and 2.5 for highly alkaline and less alkaline waters, respectively.

The dosage of reclaimed alum required for good coagulation was determined by the same methods used to determine the required dosage of commercial alum. These methods included the well known jar tests and the neces-

TABLE 3

Results With Commercial and Reclaimed Alums

Water Tested	Color*	pH	Total Alkalinity (as $CaCO_3$) ppm	Total Hardness (as $CaCO_3$) ppm	Noncarbonate Hardness (as $CaCO_3$) ppm
Raw water†	50	7.3	106	128	22
With commercial alum‡	10	6.8	68	128	60
With reclaimed alum§	10	6.8	70	128	58

* Determined by visual comparison with the use of nessler tubes.
† Temperature, 63° F.
‡ 70 ppm alum plus 2.5 ppm silicon dioxide.
§ Plus 2.5 ppm silicon dioxide.

cylinder was approximately 0.50 of 1 per cent by weight. The aluminum oxide content varied between 0.15 and 0.20 of 1 per cent, which would be equivalent to a 0.9–1.2 per cent solution of commercial alum. These concentrated sludge samples were reacted with enough sulfuric acid to convert the aluminum hydroxide to aluminum sulfate. The amount of acid applied varied, depending on the alkalinity of the raw water. If the alkalinity content was large, a quantity of acid greater than that required for the conversion could be contained in the alum solution, decreasing the dosage of re-

sary chemical and physical tests. Comparative data are shown in Table 3.

These laboratory studies were carried out over a long period of time on the various types of water that would be encountered with the Tampa supply. The need for further studies in pilot plants was indicated.

Pilot Plant Studies

Three pilot plants were installed, two for determining the maximum thickening that could be obtained at various flow-through periods and one for recycling the reclaimed alum.

Fig. 3. Recycling Pilot Plant

After the reclaimed alum was recycled from eight to ten times, there was no indication of interference from the small amount of acid-insoluble material or any change in the chemical or bacteriologic quality of the water.

One of the thickening units was a cylindrical tank equipped with a feed well, overflow, conical sludge outlet with a timer-regulated sludge discharge, and a centrally located shaft equipped with radial arms and scraper blades for moving the settled sludge gently to the conical sludge outlet. The sludge blanket in the thickener was maintained at a constant level, with the clarified water going out the overflow. The second thickening unit was a flat-bottom rectangular tank equipped with sludge collectors and a sludge hopper. The feed was distributed at one end, and, as it flowed through the basin, the floc settled to the bottom and concentrated, while clarified water went out the overflow. This thickened sludge was raked into the hopper, from which it was discharged by a timer-controlled blowoff. Feed for the two units came from the sludge discharge of the two suspended-solids-contact basins.

After the plants were operated for several weeks, it was conclusively demonstrated that the aluminum hydroxide

sludge could be thickened to 1 per cent by weight with a 3-hr flow-through period. This solids concentration contained 0.4–0.5 of 1 per cent aluminum oxide, which is equivalent to a 2.35–2.94 per cent commercial alum solution. In a plant size operation, a greater degree of thickening than is obtained in the pilot plant should be expected, because of the greater depth and pressure and the larger sludge mass. Both sludge-thickening pilot plants are shown in Fig. 2.

The third pilot unit consisted of a 30-in. scale model suspended-solids-contact basin, with the same design as the two prototypes now in operation. This unit was first used for treating raw water with reclaimed alum. The floc from the concentrator was collected in a 50-gal tank and thickened by settling, then reacted with concentrated sulfuric acid for conversion to liquid alum. The strength of the reclaimed alum solution was adjusted in a separate tank and pumped to the raw-water influent line with a hypochlorinator. Activated silica was used as a coagulant aid, and the gravity feed

TABLE 4

Results With and Without Use of Recycling Pilot Plant

Tests	Color*	Total Alkalinity (as CaCO₃) ppm
Raw water	110	60
Pilot unit, 20-hr operation	9	15
Large plant operation	5	21
Raw water	130	50
Pilot unit, 8–10 recycles	18	5
Plant operation	6	13

* Determined by visual comparison with the use of nessler tubes.

came from a constant-solution-feed device made from a 5-gal carboy. Jar tests were used to adjust the dosage as the chemical composition of the raw water changed.

After the reclaimed alum was recycled from eight to ten times, there insoluble material at a satisfactory level. The pilot plant used for recycling the reclaimed alum is shown in Fig. 3. Table 4 gives the results at the startup and after recycling had occurred approximately eight to ten times.

TABLE 5

*Analysis of Waters in Various Stages of Treatment**

Duration of Run	Color†	Total Alkalinity (as CaCO₃) ppm	Total Hardness (as CaCO₃) ppm	pH	Noncarbonate Hardness (as CaCO₃) ppm
			Raw Water‡		
	65	68	104	7.3	36
		With Reclaimed Alum—No. 2 Mixing Basin			
8 AM–4 PM	15	43		6.1	
4–12 PM	15	26		5.9	
12 PM–8 AM	19	28		5.9	
		With Commercial Alum—Solids-Contact Basin			
8 AM–4 PM	7	45		6.4	
4–12 PM	8	40		6.4	
12 PM–8 AM	8	45		6.5	
			Finished Water		
8 AM–4 PM	4	49	110	8.6	61
4–12 PM	3	56	122	8.5	66
12 PM–8 AM	5	58	126	8.2	68

* Average values given.
† Determined by visual comparison with the use of nessler tubes.
‡ The temperature of the water was 79° F.

was no indication of interference from the small amount of acid-insoluble material or of any change in the chemical or bacteriologic quality of the water. If the insoluble material should cause any interference in plant scale operation, it would only be necessary to blow down, or waste, the amount of sludge necessary to maintain the

Large Plant Operation

As the Tampa plant is very flexible—consisting of a conventional plant and two 15-mgd suspended-solids-contact basins that can be operated in series with, or parallel to, conventional settling basins—it seemed to be entirely feasible to try the reclaimed alum in a large plant operation. The re-

claimed alum was used in one contact unit, and commercial alum was used in the other as a control. Aluminum hydroxide sludge from one of the solids-contact basins was pumped to one of the mixing basins in the conventional part of the plant, which was temporarily out of service. This basin

out the overflow to the sedimentation basins for recovery. This clarified water has the same chemical characteristics as the treated water and amounts to about 3 per cent of the flow through the solids-contact basins. Thus, at a rate of 30 mgd, there would be available approximately 900,000 gal of
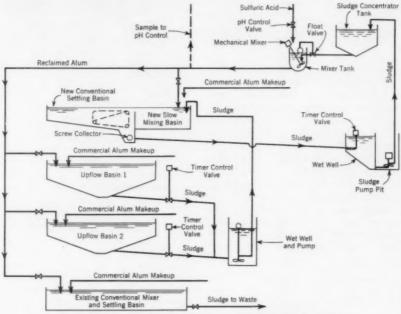
Fig. 4. Operation for Processing Aluminum Hydroxide Sludge

With this installation, most of the sludge, and some of the treated water used to convey it from the process, can be recovered.

has a capacity of 138,000 gal, sufficient for an ample supply of sludge for an operation lasting several hours. The basin influent valve was closed, the effluent valve open. The partly concentrated sludge was pumped to the basin, allowing the sludge to settle and thicken while the clarified water went

alum-treated water for recovery. The cost of chemicals for treating this water is $12–$25 per million gallons.

When the sludge level in the mixing basin neared the overflow, the sludge was pumped from the bottom to a 100-gal reaction tank, to which 66-deg-Baumé sulfuric acid was added to con-

TABLE 6

Quantity and Cost of Materials Used

Item	Amount
Quantity of water treated with reclaimed alum—*mil gal*	15.3
Quantity of sulfuric acid used—*lb*	6,632
Total cost of acid (at $19.48 per ton)—*$*	64.60
Cost of acid—*$/mil gal*	4.22
Quantity of water treated in solids-contact basin—*mil gal*	19
Quantity of commercial alum used —*lb*	15,930
Total cost of commercial alum (at $38.08 per ton)—*$*	303.31
Cost of commercial alum—*$/mil gal*	15.96
Savings with reclaimed alum—*$/mil gal*	11.74
Total savings with reclaimed alum—*$*	179.62

vert the aluminum hydroxide to aluminum sulfate, with the use of manual pH control. This reclaimed alum solution was then pumped back to the point of application for reuse.

The length of each run was 3–24 hr, depending on the type of mechanical difficulties encountered in this improvised manual operation. A large variation in the sludge concentration was noted. The variation resulted from the wide, flat bottom of the basin and the rapid thinning of the sludge in the vicinity of the pump suction. This necessitated moving the pump suction over the bottom of the basin to obtain as much of the concentrated sludge as possible. The variation in sludge quality necessitated considerable adjustment in the acid feed. Some of the sludge samples collected had a solids concentration as large as 2 per cent by weight and contained 0.8–0.9 per cent aluminum oxide, which is equivalent to a 4.7–5.3 per cent alum solution. This type of operation was continued over a long period of time on the various types of water, with part of the plant used as a control on commercial alum. The water was tested every 2 hr during a 24-hr run. Average values for each 8-hr shift are shown in Table 5. Comparative cost data are given in Table 6.

Proposed Facilities

The next expansion at the treatment plant will include 20-mgd conventional settling basins. The final decision is to use these basins to recover the aluminum hydroxide sludge from the other treatment units, instead of installing a separate thickener. With this installation, most of the sludge, and some of the treated water used to convey it from the process, can be recovered, and, at the same time, the raw water entering the basin can be treated with the normal alum dosage.

The proposed facilities will be equipped with sludge collectors. The sludge from the existing basins will be pumped, with the raw water, to the new mixing basin, where the alum dosage will be applied. The floc will go through the flocculating basins and then to the sedimentation basin, where it will settle and concentrate. This thickened sludge will be raked into a hopper by the collectors and pumped to the reaction tank, where the concentrated sulfuric acid will be added. The process will be regulated by a

TABLE 7

Cost of Chemicals at New Plant

Item	Amount
Cost of commercial alum—*$/ton*	38.08
Cost of sulfuric acid—*$/ton*	19.48
Cost of 1,180 lb of sulfuric acid—*$*	11.49
Savings with reclaimed alum—*$*	26.59

pH control similar to the system now used to control the liquid alum at the plant. From this reaction tank, the reclaimed alum will be conveyed back to the treatment process for reuse. Figure 4 shows the operation involved in processing the aluminum hydroxide sludge.

The amount of 93 per cent sulfuric acid with a 90 per cent yield needed to produce 1 ton of 17 per cent reclaimed alum is calculated as:

$$H_2SO_4 = \frac{3 \times 98 \times 2,000}{594} = 990 \text{ lb}$$

$$\frac{990 \times 100}{93} = 1,065 \text{ lb}$$

$$\frac{1,065 \times 100}{90} = 1,183 \text{ lb}.$$

The cost of chemicals needed is shown in Table 7.

Conclusion

Although some difficulties may develop in the recovery process described and in the mechanical methods employed, laboratory and plant scale tests fully prove that the process is practical. Refinements and automatic control features may be added later if a need for them is demonstrated. It should be noted that when the plant is operating principally as a softening plant, aluminum hydroxide sludge is not available for the recovery process.

References

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Use of Sodium Hexametaphosphate in Manganese Stabilization

George L. Illig Jr.

A contribution to the Journal by George L. Illig Jr., Asst. to the Vice-Pres., Industrial Chemical Sales Div., Hagan Chemicals & Controls, Inc., Pittsburgh, Pa.

PROBABLY the least publicized water treatment application of glassy sodium phosphate (sodium hexametaphosphate) is in the stabilization of dissolved manganese. Nevertheless, at least 40 municipal and industrial water plants have used this method of overcoming an otherwise difficult water treatment problem. Waters containing dissolved manganese almost always contain dissolved iron, and it is fortunate that there are marked similarities in the methods of stabilization of both substances with glassy phosphate. For example, the glassy phosphate must be well mixed with the water before the manganese and iron are oxidized and start to precipitate. In addition, the glassy phosphate must be fed in definite proportion to the iron and manganese content. This usually means a ratio of 2 parts glassy phosphate for each part manganese and iron (combined), although under some conditions a 4:1 ratio is required. As a general rule, if the iron and manganese content is less than 1 ppm, phosphate dosage of not less than 2 ppm should be used. This dosage allows for adsorption of a portion of the phosphate on the metal surfaces of the distribution system piping.

Control of Color Development

Proper stabilization of iron and manganese with glassy phosphate prevents color from developing in the treated water. It should be emphasized, however, that glassy phosphate does not prevent oxidation of manganese and iron, even though it must be added before they have begun to precipitate. In iron stabilization it has been found that an almost colorless iron phosphate complex is formed. Although the iron phosphate is not visible to the naked eye, it is present as a well dispersed colloidal suspension. The dispersion prevents growth to sufficient size to result in actual precipitation. Thus, for all practical purposes, the iron remains in solution and does not form the characteristic rusty red iron oxide color. The same general mechanism also appears to take place with dissolved manganese, thus preventing development of "black water."

Iron and Manganese Oxidation

Both manganese and iron are subject to oxidation and precipitation by iron bacteria, even in water treated with glassy phosphate. As iron bacteria are usually present in waters containing manganese and iron, adequate

disinfection of the water supply is necessary if the maximum benefit is to be obtained from the use of glassy phosphate. Consequently, in treating well water that is pumped directly into the distribution system, the phosphate solution is usually fed directly to the well through a drop line extending to

Manganese oxidation differs from iron oxidation in at least two significant respects. Whereas iron is readily oxidized by aeration or chlorination at pH levels about 6.0, manganese is much less susceptible to oxidation at pH levels below 8.0. This suggests that the point of phosphate feed is not
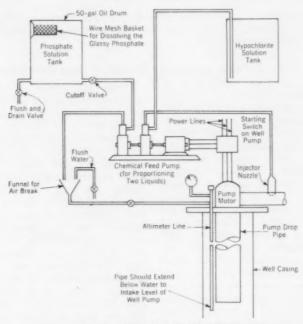
Fig. 1. Typical Glassy Phosphate Feeding System

Because concentrated sodium hexametaphosphate solutions are corrosive to iron or steel in the absence of enough calcium for the formation of a protective film, corrosion-resistant equipment should be used.

the suction level of the well pump. Chlorine or hypochlorite should be fed on the discharge side of the well pump for control of iron bacteria. A schematic drawing of a typical chemical feeding system for dissolved manganese and iron stabilization is shown in Fig. 1.

quite as critical in stabilizing manganese as it is in stabilizing iron. In fact, success has been obtained in several instances by feeding glassy phosphate to the filtered water to tie up residual manganese even though the water was previously aerated or coagulated in a water treatment plant.

Manganese is also much more prone to catalytic oxidation than iron. Old manganese deposits in the distribution system tend to speed up the oxidation and precipitation of dissolved manganese. Therefore, in a system containing such manganese deposits, a higher feed rate is required than in a clean system. This higher initial feed rate, when used in conjunction with a systematic hydrant-flushing program, assists in cleaning out any loosely adhering manganese deposits and serves quickly to saturate the surfaces of the distribution system with glassy phosphate. This, in turn, provides a means of quickly stabilizing the manganese so that it can be carried through the system without precipitation.

Successful Applications

Although there is theoretically no limit to the amount of manganese that can be stabilized if the proper amount of glassy phosphate is fed, there is an economic limit to the amount of stabilization a water utility can afford. This limit varies between 1 and 2 ppm for the larger plants to a maximum of 5 ppm for smaller plants. Presented below are four case histories of the successful application of sodium hexametaphosphate for manganese stabilization at communities in various parts of the United States. These are but a few of the many successful examples that could be cited.

Case History 1

An Ohio River Valley community had recognized the possibility of using glassy phosphate for manganese stabilization several years before, but the first brief field tests were carried out in the late summer of 1938. The city pumped its supply directly into the distribution system from two wells, one

of which produced a water containing 8 ppm manganese. As the other well was virtually free of manganese, the water reaching the consumer contained approximately 4 ppm manganese. An initial attempt to stabilize the manganese by the application of only 2 ppm glassy phosphate was completely unsuccessful. This is not surprising, as records also disclosed that the glassy phosphate had not been applied until after the water from both wells had been mixed together and chlorinated. After further laboratory

TABLE 1

Analyses of Water Treated with Sodium Hexametaphosphate, Case History 1

Substance	Concentration—*ppm*	
	Combined Well Waters Entering Distribution System*	Near End of Distribution System†
Bicarbonate	96	94
Chloride	15	15
Iron	0.3	0.2
Manganese	2.1	2.0
Hardness (as $CaCO_3$)	205	206
Metaphosphate	6.5	3.6
Orthophosphate	0.6	1.0

* pH 6.8.
† pH 6.9.

study and additional field testing, glassy phosphate treatment was successfully reinstated in this system in the summer of 1941. At that time, the combined well waters had a manganese content of only 2.1 ppm. A glassy phosphate feed rate of 5 ppm was recommended and found to be adequate. The manganese-bearing well water contained approximately 5 ppm of dissolved manganese, and all of the glassy phosphate was fed to that well at a dosage of 10–15 ppm, depending on the pumping rate. Table 1 con-

tains typical analyses of the combined well waters at the beginning of the distribution system and at a point well out in the distribution system. At the time that these samples were taken the manganese-bearing well was providing roughly 40 per cent of the total water.

Prior to the establishment of glassy phosphate treatment, hundreds of consumer complaints were received each week because of unsightly staining of clothes washed in the manganese-bearing water. During the early stages of treatment, the system was flushed systematically in order to clean out old deposits as rapidly as possible. Except during the flushing program, no further consumer complaints were received even during the early weeks of treatment. Subsequent experimentation at lower feed rates indicated that complaints of staining arose whenever the feed rate dropped below 4.5 ppm.

This treatment provided satisfactory results over a 4-year period with glassy phosphate consumption varying from 75 to 200 lb/day, depending upon the demand of the community. Treatment was finally interrupted when new manganese-free wells of increased capacity were added to the system. Nevertheless, in the intervening years it has occasionally been necessary to restore the old manganese-bearing well to service during periods of peak demand. On such occasions the utility has wisely resumed the feed of glassy phosphate to the old well and avoided renewed complaints of "black water," manganese-stained laundry, and discolored plumbing fixtures.

Case History 2

A municipality in the western United States used a large number of wells scattered throughout the community, but manganese troubles were mostly confined to a particular well field consisting of seven active wells. Water from this group of wells was brought together at a common point where glassy phosphate was added before the water was aerated or chlorinated. Following addition of glassy phosphate, the water was chlorinated, aerated, collected in a reservoir, and pumped into the system.

The pH of the mixed well waters was 8.0 and the average manganese content of the mixed well water at the point of phosphate addition was only 0.2 ppm, but, in the absence of phosphate treatment, complaints of "black water" were frequently encountered. Experimentation showed that a minimum feed rate of 2 ppm was necessary to stabilize the 0.2 ppm of manganese usually present in the water. Tests run on an individual well also revealed that a feed rate of 3.0 ppm or more would be required to stabilize well waters containing 0.5–1.0 ppm with the rate of superchlorination that was practiced.

Treatment continued over a 10-year period at feed rates between 2 and 3 ppm. As additional wells were added to the system, it was found that most of them were free of dissolved manganese, although it was necessary to feed glassy phosphate to one of the new wells containing 0.2 to 0.3 ppm manganese. With the addition of newer wells, the resultant intermixing of water, and the flow reversals that occur in this system, there have been occasional complaints of dirty water. The water department officials of this community claim that glassy phosphate treatment has helped to keep such complaints to a minimum through

the years. The glassy phosphate consumption has been 100–125 lb/day.

Case History 3

At a city that obtains its water from a group of radial collector wells located along the Ohio River, serious manganese problems were encountered. The well water was pumped to a treatment plant before entering the distribution system. Treatment consisted of prechlorination, aeration, sedimentation without coagulation, and, with less than 30-min retention, filtration through rapid sand filters. In an attempt to control corrosion, lime was then added to the clear well to hold the pH between 8.0 and 8.2. Postchlorination was then used to hold a 0.4-ppm chlorine residual in the finished water.

Although a portion of the manganese was removed by the combined effect of prechlorination, aeration, and filtration, plant limitations precluded the utilization of lime or permanganate for more efficient manganese removal. The original plans were to discontinue lime addition at the clear well and to feed glassy phosphate directly to the wells. Two of the collector wells, however, contained such an unusually high amont of manganese when they were first pumped that a mixture of water from the three wells prior to treatment contained 7.2 ppm dissolved manganese. This made it necessary to pump two of the wells to waste for several weeks.

Eventually, with use of only two wells at a time, the manganese content of the raw water varied from 2 to 3 ppm and the iron content was 0.1–0.4 ppm. Thus, a glassy phosphate feed rate of 5–6 ppm would have been required to stabilize the iron

and manganese in the raw water. In an attempt to hold treatment costs to a minimum, it was decided to feed lime to the water ahead of the filters, thus reducing the amount of manganese in the finished water, and to apply glassy phosphate to the clear well to tie up the manganese still in solution. With this treatment scheme the manganese content of the finished water stabilized at 0.4–0.5 ppm and a glassy phosphate feed rate of 2 ppm was recommended It originally was proposed to use a high phosphate feed rate during the first 2 weeks of treatment in conjunction with a systematic hydrant-flushing program before cutting back to the recommended 2-ppm feed rate. The high phosphate feed rate was intended to saturate the system thoroughly as rapidly as possible and to assist in the removal of loosely adherent sludge or manganese deposits known to be present in the system. However, under the critical operating conditions of the plant, the water department personnel decided to avoid shocking the system.

Field engineers of the phosphate supplier then recommended that treatment be started at a 0.6-ppm level and that the feed rate be increased gradually over a period of a month up to a 2.0-ppm level. After the 2.0-ppm level was reached, the system was flushed systematically. No complaints of "black water" were received at any time during the critical startup period. This was true even though such complaints had been frequent prior to the initiation of glassy phosphate treatment. Approximately 1 month after phosphate treatment was started, the three collector wells were pumped simultaneously with the manganese content of the untreated mixed well

waters at 2.6 ppm and 0.4 ppm in the finished water. Samples taken several weeks later showed 2.8 ppm manganese in the mixed raw well water and 0.5 ppm in the finished water. After the second month of treatment, samples taken from the distribution system showed that adequate metaphosphate residuals were present with virtually no indication of manganese deposition (Table 2).

Fifteen months after treatment was started, one water department official stated that "black water" complaints

taining 0.3–0.5 ppm manganese in addition to 0.2–0.4 ppm iron. Based on a maximum combined iron and manganese content of 0.9 ppm, a glassy phosphate feed rate of 3.5 ppm was recommended. The glassy phosphate was fed to a common header fed by several different wells and chlorine applied to the discharge side of the pumps at the pumping station.

When treatment was started in this system, which had a long history of "black water" difficulties, a systematic hydrant-flushing program was recom-

TABLE 2

Analyses of Water 2 Months After Start of Treatment With Sodium Hexametaphosphate, Case History 3

Substance	Concentration—*ppm*			
	Mixture of Raw Well Waters*	Finished Water at Clear Well†	Midpoint of System‡	Near System Extremity§
Bicarbonate	104	102	105	
Chloride	54	53	54	51
Iron	0.6	0.3	0.2	0.2
Manganese	2.8	0.5	0.4	0.4
Hardness (as CaCO₃)	243	240	242	239
Metaphosphate	—	1.6	1.0	0.5
Orthophosphate	—	0.5	1.0	1.4

* pH 6.9.   † pH 8.2.   ‡ pH 8.1.   § pH 8.3.

were a thing of the past, thanks to glassy phosphate treatment, even though the finished water consistently contained 0.4–0.5 ppm manganese. Treatment results were uniformly good at this plant for more than 5 years, until the plant was completely renovated and provisions were made within the plant for manganese removal by chemical precipitation.

Case History 4

One Northeast municipality found it necessary to treat well waters con-

mended in conjunction with an initially high glassy phosphate feed rate of 5 ppm. Because of the water shortages, however, it was not possible to carry out the hydrant-flushing program until almost 8 months later. Nevertheless, an improvement in the quality of the water was noted shortly after the treatment was started, and the local water department personnel were favorably impressed with the results of glassy phosphate treatment.

Before the hydrant-flushing program was begun, the phosphate feed

was raised to about 5 ppm to assist in the cleanout program. The water department first started its flushing operation during regular working hours, but complaints of dirty water immediately ensued. Therefore, to minimize complaints, the flushing program was conducted only during the night hours, starting after 11:00 PM, and no additional complaints were received by the department.

Table 3 shows typical water analyses of the mixed well waters at the pumping station and of samples from

Conclusion

In the Pittsburgh metropolitan area alone there are several communities using manganese-bearing waters as a primary source of supply. Several of the smaller communities feed glassy phosphate directly to their wells at feed rates varying from 3 to 5 ppm. At least two large water supplies have also occasionally used glassy phosphate to tie up dissolved manganese that has leaked through the filters when the manganese content in the raw water was unusually high. As

TABLE 3

Typical Analyses of Water Treated With Sodium Hexametaphosphate, Case History 4

Substance	Concentration—*ppm*			
	At Pumping Station*	Midway in System†	Further on in System*	Near End of System†
Bicarbonate	30	32	32	32
Chloride	13	12	12	13
Iron	0.4	0.4	0.4	0.4
Manganese	0.5	0.5	0.5	0.5
Hardness (as $CaCO_3$)	43	43	43	46
Metaphosphate	1.8	1.1	1.0	0.7
Orthophosphate	0.3	0.5	0.5	0.4

* pH 6.5.
† pH 6.4.

the distribution system. Although the phosphate feed rate appeared to be lower than the normal 3.5 ppm at the time of sampling, the iron and manganese were well stabilized as the water moved through the system. Data are shown for a point midway through the system, a point still farther away from the pumping station, and another point near the end of the distribution system.

This water plant has continued to use glassy phosphate for manganese stabilization for more than 10 years, with continuing good results.

manganese leakage through the filter amounts to less than 1 ppm, stabilization can be achieved safely at a metaphosphate feed rate of 2 ppm.

A fairly reliable indication of the effectiveness of glassy phosphate for manganese stabilization can be obtained in a relatively few weeks. In extreme cases of "black water," improvement should be noticed almost immediately. Fewer complaints should be received, particularly after loosely adhering manganese deposits have been cleaned out. Furthermore, water utilities that conduct systematic hydrant-flushing

programs on an annual, semiannual, or even a quarterly basis will find that hydrants clear up much more quickly than formerly and, in most cases, require less frequent flushing. Although attempts have been made to measure the extent of manganese deposition in systems when the average manganese content is as low as 0.1–0.3 ppm, a more practical method is to check manganese concentrations of the water at various points throughout the distribution system. After a system using water properly treated with glassy phosphate has been cleaned of old, loose deposits, the level of manganese should be the same at the various sampling points throughout the system.

It has been shown that glassy phosphates can be utilized successfully to stabilize dissolved manganese under different operating conditions and in waters of markedly different chemical characteristics. Consideration should be given to this treatment method whenever engineering studies are undertaken to find an economical solution to the manganese control problem.

Correction

The article by Frank C. Roe, "Experiences in the Use of Porous-Plate Filter Bottoms" (May 1960 JOURNAL, Vol. 52, pp. 653–659), contained a misleading typographical error. On p. 658, in col. 1, line 24, the phrase "depending on quality" should read "depending on *quantity*."

Organic Fouling of Anion-Exchange Resins

————Norman W. Frisch and Robert Kunin————

A contribution to the Journal by Norman W. Frisch, Research Engr., and Robert Kunin, Research Assoc., Rohm & Haas Co., Philadelphia, Pa.

THE availability of anion-exchange resins of the strongly dissociated type has made possible the economical production of highly deionized water. The quaternary resins are usually sufficiently stable that, at temperatures below approximately 50°C, high cumulative throughputs of high-quality water are obtained. Under such conditions, water quality deteriorates very slightly, usually exhibiting only small increases in the leakage of weakly dissociated acids—that is, silicic and carbonic acids. The degradation is known to be dependent upon temperature and the presence of oxygen and oxidizing agents, and it is capable of being catalyzed by metals, notably copper.[1]

In some localities it has been noted that water quality decreases rapidly when there is a low rate of irreversible degradation of functional sites. A decrease in effluent pH—from 7 in monobed systems and from the range 7–10 in two-bed systems—occurs. Effluent pH continuously decreases, often leveling off at pH values in the range 4–6. Examination of treated water reveals the presence of material with high chemical oxygen demand (COD). Often color bodies are detectable in treated water after considerable concentration by evaporation. Also, color bodies are evident in the waste caustic regenerant from these anion-exchange beds. Decreases in water quality usually become more severe, effluent quality often being limited to a resistivity of several hundred thousand ohm-centimeters. Eventually, the operating capacity of the bed is severely reduced. The occurrence of fouling is most frequently noted with surface waters, and it is not limited to highly industrialized areas.

Examination of anion-exchange resins in plants producing an effluent of limited quality reveals the presence of dark-colored materials on the resin. It is evident that the resins were exchanging organic acids present in the feedwater until their capacity for further exchange was exhausted. Leakage of these acids then occurred, with some elution of loosely bound organic acids during the regeneration operation. Exhaustive studies were conducted in the laboratory of the authors' firm in order to ascertain the nature of the foulant, means of estimation of its concentration in the feedwater and on the contaminated resin, and methods of reducing fouling of the resins.[2,3] All resins used in the tests are identified by letter symbol below.*

Resin	Trade Name
A	Amberlite IRA-400
B	Amberlite IRA-120
C	Amberlite IR-45
D	Amberlite IRC-50
E	Amberlite IRA-401
F	Amberlite IRA-402

* All products of Rohm & Haas Co., Philadelphia, Pa.

Resin Performance

The initial hypothesis that contaminants were present in the feedwater was supported by tests involving the feeding of water containing tea, extracts of peat moss, and lignin derivatives. Even at low concentrations of tea (Fig. 1) an effluent of decreased quality was produced from monobeds, which is characteristic field behavior for water with a high COD. The pH and quality patterns are heavily dependent on the concentration of organic
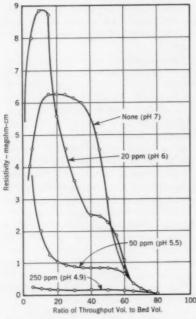
Fig. 2. Behavior of a Mixed Bed Containing Strongly Acidic and Strongly Basic Resins

The potassium permanganate demand of the raw water was 10–12 ppm. The similarity between the curve for filtered raw water and the curve for 50 ppm tea in Fig. 1 should be noted.

material in the feed. In most of the tests of this type, however, only limited loss of functional capacity could be detected at the point at which decreased water quality was noted. This result is not in agreement with those of examinations of resins fouled in the field. No doubt, the difference is due to the vast difference in the time patterns. In laboratory studies, fouling is usually investigated with high concentrations of foulants in contact with the resins for a limited time. In the field, long contact time presents more opportunity for diffusion of the foulants through the resin gel and, therefore, for binding of foulants at a significant number of exchange sites.

The data of Bogers,[1] reproduced in Fig. 2, show the behavior of a mixed-bed unit containing strongly acidic and strongly basic resins deionizing raw water with a potassium permanganate demand of 10–12 ppm. During a small portion of a cycle, water resistivity was in excess of 1 megohm-cm.

Fig. 1. Performance of Monobeds With Water Containing Varying Concentrations of Tea

Curves are labeled to show nominal tea concentrations and average pH values. As can be seen, an effluent of decreased quality was produced even at low tea concentrations, thus indicating the presence of foulants in the feedwater.
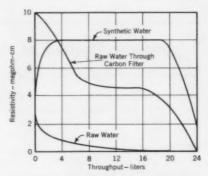
With a synthetic water prepared from distilled water and containing inorganic salt equivalent to that of the raw water, a high-quality effluent was obtained over the normal cycle. Passage of raw water through a carbon filter prior to passage through the mixed bed resulted in performance intermediate between that of the raw water and that of the synthetic water. The curve is of the same shape as the performance curve for a nominal tea concentration of 50 ppm (Fig. 1). This behavior reveals that at these intermediate organic concentrations the resins possess limited capacity for removal. As soon as the exchange sites are exhausted, abrupt leakage occurs. The levels of the specific-resistance data in each instance are closely related to the pH, which indicates that the leakage of organic material contributes largely to the conductivity values.

Analysis of Water

A simple test for determining the organic content of water involves various COD tests using permanganate or dichromate. This test gives little indication of the nature of the contaminants, but it serves to give some comparison of the relative concentration of organics in a stream during various stages of processing. Bogers [1] found that the raw water he investigated had a permanganate demand of 10–12 ppm, but after passage through a cation exchanged bed it was about 8 ppm; filtration was postulated to be the cause of this reduction. Initially the mixed-bed effluent had a demand of 4 ppm but it increased to 8 ppm, indicating complete leakage at the end of the cycle. Chloride leakage was only about 0.1 ppm.

Other tests of limited specificity include chlorine demand, nessler reagent for ammonia and other nitrogen-containing compounds, Biuret test for proteins, and phosphotungstic acid may be used for the determination of tannins. In practice, in addition to finding high COD values, analysts often note negative or inconclusive results with these procedures. One difficulty involves the limited concentrations of foulants being investigated. Obviously, concentration techniques are useful in this instance. Two approaches are worthy of investigation: (1) direct concentration of raw and deionized water by evaporation, and (2) use of solid adsorbents.

In the research reported here, direct water evaporation was practiced in platinum ware with a nitrogen gas

TABLE 1

Analysis of Deionized Philadelphia Water*

Item	Method	Concentration *ppm*
Total solids	gravimetric	0.91
Volatile organic matter		0.72
Inorganic salts	by difference	0.19
SiO₂	colorimetric	0.0004
	gravimetric	0.0086

* Specific resistivity 7.5 megohm-cm.

stream to accelerate transfer of water to the vapor phase. By using a sample of 25 liters or more, it was possible to obtain an accurate evaluation of the content of the deionized water sample. A typical analysis of deionized water from a monobed containing Resin A after 42 cycles on Philadelphia city water derived from the Delaware River is shown in Table 1. Obviously, the major leakage is that of organic materials. At these levels no noticeable difference was detectable in the ultraviolet spectra of the deionized water and distilled water. Under similar conditions, effluent from a monobed treating Delaware River water, which

was subjected to a coagulation and filtration step only, was found to contain 1.40 ppm organic matter, about twice that of the unit treating Philadelphia water. The inorganic solids from both beds were essentially the same. The organic residue was dark brown, possessed a distinct caramel odor, and, upon ignition, yielded an odor similar to that of protein decomposition products.
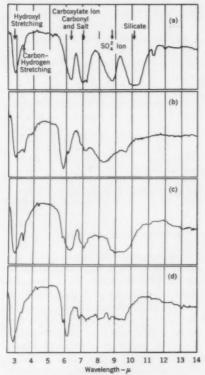

Fig. 3. Infrared Spectra of Deionized and Raw Foulants, a Strong Tea Solution, and an Infusion of Leaves

Parts a, b, c, and d are spectra for the raw foulant, deionized foulant, a strong tea solution, and an infusion of leaves, respectively.

Solid adsorbents serve admirably to concentrate traces of organic acids in water. One recommended procedure involves passage of the upflow stream through an activated-carbon bed. The contaminants are readily recovered from the carbon by solvent extraction. Chloroform and alcohol appear to be preferred solvents. Their use results in relatively high recovery of adsorbates, although quantitative recovery of phenolics is not readily obtained. After evaporation of solvents, the recovered adsorbates may be fractionated by a variety of methods. A simple scheme, involving solubility in water or ether or extractability in acidic or basic solutions, is outlined in a manual of the Robert A. Taft Sanitary Engineering Center.[7] Final characterization of the fractions may be obtained by infrared techniques.

Although this technique is exceedingly useful for the overall study of organic acid in water, this article is chiefly concerned with those organic (acidic) components which are responsible for the fouling of anion exchangers. A simple and direct approach to the latter involves the use of the anion-exchange resins in the field as solid adsorbents for concentration of the traces of organic foulants found in the water supply. Thus, either pilot installations or full-scale units may serve for this end. Although the caustic regenerant is generally ineffective in eluting these organic foulants, it was noted early in this study[2] that sodium chloride, at concentrations of 5–10 per cent, was capable of producing highly colored effluents from fouled anion exchangers. Also, higher peak concentrations of color bodies were obtained by use of iodide on nitrate salts.[2] The latter ions are highly selectively exchanged on strongly basic ion-exchange resins.

TABLE 2

Analysis of Recovered Foulant

Element	Concentration per cent
Nitrogen	2.0
Sulfur	0.7
Phosphorus	1.3
Iron	0.09
Copper	0.09
Magnesium	0.0009

Thus it may be inferred that at least a portion of the fouling substances are bound by normal ion-exchange forces. Elution at higher temperatures—that is, 110°–130°F—produced a higher peak concentration of color bodies. It appears that this elution is gel diffusion controlled.

Examination of the eluted foulant was made in the laboratory. The solution was recovered from a commercial unit which treats a surface water, and, after brining with an 8 per cent sodium chloride solution at 110°F, the resulting solution was highly colored and acidic and possessed an odor similar to that of molasses. Prior to characterization of the foulant, it was deionized by treatment with a monobed mixture of Resin B, a sulfonic acid resin in the hydrogen form, and Resin C, a weakly basic resin in the free-base form. The latter possesses a limited capacity for large, weakly acidic ions. The composite solution treated in this manner contained 1.56 per cent of organic matter. Titration with standard alkali revealed the presence of a fairly strong acid with a pK value of 3.7. The solute was soluble in acid and base. Absorption increased in alkaline solution; this behavior is typical of humic acids. An analysis of the dried solids is given in Table 2.

By evaporation of a different batch of deionized foulant solution obtained from the same source, potassium bromide discs were prepared containing 0.5 per cent solids. Infrared spectra

were determined on the deionized foulant, the raw foulant, a strong tea solution, and an infusion of leaves. In Fig. 3, the infrared spectra of these four systems are shown. The spectrum for the raw foulant shows a strong band at $6.4\,\mu$ which is assigned to the carbon-oxygen stretching band of a carboxylic acid salt. A significant content of inorganic salts is present. Sulfate and silicate are indicated on the spectrum. Fig. 3 (b), prepared from the deionized foulant, indicates essentially complete removal of the sulfate and silicate bands. A great reduction of intensity of band of A attributed to the presence of a carboxylic salt is consistent with the deionization to yield a free carboxylic acid. The band at $5.8\,\mu$ can be confidently assigned to the carbon-oxygen stretching band of a carboxylic acid. No evidence of the presence of sulfonates was noted.

The comparison spectra are from natural products of a very complex nature. In general, the spectra of solids from tea and an infusion of dead leaves (Fig. 3c and d) are much the same and are rather featureless. Both appear to contain significant quantities of carboxylic acid salts. It is concluded that the foulant is not unlike acidic material capable of being leached from decaying vegetable matter.

Further examination of the nature of the deionized foulant was made. The solution had a high COD. The foulant was capable of being adsorbed largely irreversibly on Resin A (OH

TABLE 3

Ion-Exchange Resin Scavengers

Resin	Form	Description
B	Ca^{++}	strongly acidic
D	Ca^{++}	weakly acidic
	Pb^{++}	weakly acidic
E	Cl$^-$	porous; strongly basic

form). Two precipitants for the concentrated (1.56 per cent) foulant were found; lead salts and Hyamine 1622, octylphenoxyethyoxyethyl benzyl dimethyl ammonium chloride, were effective precipitants. At dilutions of 60 ppm, however, neither substance was particularly effective. At these reduced concentrations it was expected that the most effective removal techniques would involve a fixed-bed adsorbent process. Consequently, several ion-exchange resins were investigated (Table 3).

Figure 4 depicts the leakage behavior of these resin beds. Of the cation resins the lead salt of Resin D appeared to be most effective. Calcium salts of the weakly or strongly acid resins were appreciably less effective. Resin E (Cl form) was as expected, an effective scavenger, exhibiting less than 0.5 per cent leakage over 40 bed volumes examined, at which point operation of the column was halted. The
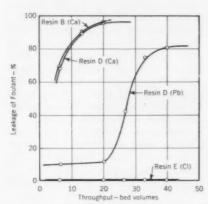
Fig. 4. **Leakage Behavior of Four Scavenger Beds**

As can be seen, the Cl form of Resin E (a strongly basic resin) was the most effective scavenger, exhibiting less than 5 per cent leakage over 40 bed volumes examined.
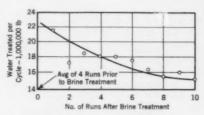

Fig. 5. **Effect of Brine Treatment on the Performance of a Commercial Ion-Exchange Installation**

Over ten cycles a 40 per cent improvement in operating capacity was recorded.

technique of using a porous, strongly basic resin as a scavenger was investigated further. The hypothesis behind this approach was that the chloride form of the porous anion-exchange resin is an effective scavenger for organic materials in water and that the use of a relatively small bed, which could be frequently regenerated with warm 5 per cent brine, would be an inexpensive and convenient method of preventing the buildup of foulants in both the caustic regenerated resin and upon itself. The use of salt regeneration on the caustic regenerated resin has been shown to be a useful technique for increasing operating capacity in plants suffering from fouling. Figure 5 shows the performance of a commercial ion-exchange installation, which had been fouled with organic matter, after a brine treatment. Over 10 cycles an improvement of about 40 per cent in operating capacity was noted. This technique does not appear to be completely acceptable in the field for several reasons. The use of this warm-brine treatment for removal of organic matter is not completely effective in restoring all of the fouled capacity because the utility of frequent brining is not appreciated until capacity is severely reduced by fouling. At that

point it is difficult to achieve complete regeneration of fouled sites. From the results of the laboratory studies and examination of field samples, it was concluded that the frequent use of relatively low concentrations of brine is more effective in keeping the resins relatively free of organic foulant than infrequent use of high brine concentrations. Again, a diffusion mechanism appears to be indicated. Once a significant portion of the resin gel has been penetrated radially, extremely long elution periods are required for the removal of the foulants. Also the brine treatment generally requires that subsequent caustic regeneration be practiced at increased levels because of the high chloride selectivity of these resins. The increase in down time because of the brining period, rinses, and increased caustic regeneration may be such that continuous deionization of raw water is not possible. For these reasons the chloride form of Resin E was investigated in a pilot plant as a scavenger resin to protect a monobed

Fig. 7. Result of Oxidation of a Sample of Resin A After 110 Cycles of Treatment

The technique used for estimation of organic matter was differential oxidation of organic matter by 0.05N ceric sulfate in 1.8N sulfuric acid (at 25°C).

resin. Figure 6 shows the comparative water quality of four different arrangements of ion-exchange resins. The monobed operated with Resin E (Cl form) scavenger performed at the highest quality level. Another containing Resin E performed nearly as well, although the reduction in capacity was greater. Under the severe conditions involved in plant water treatment, a monobed containing Resin A exhibited a rapid drop in quality. It is evident that the porous Resin E (Cl form) is an effective scavenger resin which can extend the operating life of units producing water of a high quality.

Estimation of Foulants on Resins

In addition to the characterization study, techniques for the estimation of concentration of foulants on resins have been investigated. This value is significant for several reasons. It serves as a guide to the effectiveness of water

Fig. 6. Peak Water Quality of Beds Containing Four Different Anion-Exchange Resins

Curve A—city water with Resin E (Cl form); Curve B—plant water with Resin A and Resin E; Curve C—plant water; Curve D—plant water.
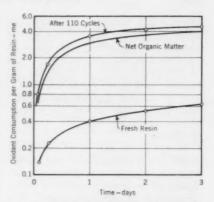
pretreatment schemes. It also can serve as an indication of when resin cleanup is warranted. As operating capacity depends upon both the extent of irreversible degradation of the hydroxide form of the resins and the extent of fouling, it is important that both resin capacity and the concentration of foulants on the resins be determined.

It was found that an efficient technique for the estimation of total organic matter on the resin was differential oxidation of the organic matter by $0.05N$ ceric sulfate in $1.8N$ sulfuric acid. The original investigation was conducted at 25°C. Figure 7 shows the result of the oxidation of a sample of Resin A removed from a monobed after 110 cycles of treating plant water. A blank was run on unused Resin A of the same

Fig. 9. **Rates of Accumulation of Organic Matter on Chloride Forms of Resin A and Resin E**

Initially, Resin E adsorbs organic acids at approximately twice the rate of Resin A.

origin. In most instances 7 days' exposure at 25°C is sufficient for complete determination of adsorbed organic matter, although modifications have been developed which permit rapid evaluation of organic matter by oxidation at high temperatures.

Figure 8 shows the accumulation of organic foulant on a porous resin in a commercial pilot plant. The curve has the characteristic shape noted in most installations which treat water with a relatively unchanging concentration of organic material. Initially, high removal is noted, but eventually appreciable leakage of organic material occurs, producing a water of limited quality.

The efficiency of scavenger beds was investigated by use of the ceric oxidation technique. The chloride forms of Resin A and Resin E were used as scavenger beds for plant water. The buildup of organic matter on the resins was followed by periodic sampling of the resin beds. Figure 9 shows the relative accumulation of organic matter

Fig. 8. **Accumulation of Foulant on a Porous Resin in a Commercial Pilot Plant**

The initial high removal, represented by the dashed portion of the curve, began to deteriorate after approximately 40 cycles, when leakage of organic material began.

on these resins; Resin E initially adsorbs organic acids at about twice the rate of Resin A. Of course, Resin E accumulates more organic matter on it during the exhaustion cycle, but also essentially all this material is eluted with brine, which is not the case for the less porous Resin A. The initial slope of the Resin E curve corresponds to about 1.8 ppm oxygen demand. As a rough guide it is believed that regeneration (brining) should be practiced on the resins before the buildups of organic matter on the nonlinear portion of the curve. This procedure permits efficient adsorption of organic material from the raw water and avoids the diffusion of excessive organic material into the resin gel. In this situation a scavenger bed roughly one-tenth the size of main anion resin unit was satisfactory.

In general no one cleanup treatment can be recommended as universally applicable. Type II resins, in particular, are not sufficiently resistant to hypochlorite treatment to permit oxidation of adsorbed organic matter. This deficiency and the limited stability of the resins to irreversible capacity loss of functional sites limit the use of these resins severely. The progress of cleanup treatments of fouled porous resin is shown on Fig. 10. The "as received" resin exhibited poor capacity. At high levels of caustic only 55 per cent of the initial sites were regenerated. Salt treatment removed about 20 per cent of the organic matter but achieved a 75 per cent recovery of fouled sites. This suggests that the foulant of relatively low equivalent weight is readily eluted whereas the organic compounds of higher molecular weight are bound too tightly to be displaced by chloride. Hypochlorite treatment at the 0.5 per cent level recov-

ered additional capacity and reduced the organic content to 35 per cent of the fouled value. Stronger hypochlorite caused some capacity loss due to attack on the functional groups.

Fouling Mechanism

The findings indicate that the foulant is a carboxylic acid, but traces of nitrogen and sulfur were also found. Bogers [1] has stated that his analysis
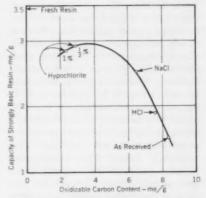

Fig. 10. Cleanup Treatment of a Foulant Porous Basin

Hypochlorite treatment (0.5 per cent solution) was the most effective cleanup method with this resin.

based on infrared spectra showed the presence of OH, NH_2, COOH, CH, and amide or a double bond; no phenyl groups could be identified.

Wilson [10] recently studied the fouling of Type II resins. By use of the ultraviolet spectra of Fuchs and Kohler,[5] he estimated the content of the raw water treated by this resin to be 1.7 ppm humic acid. The spectra possess no absorption peaks. This work is of interest in that Wilson noted that the elution of organic acids is aided by

the presence of causticity in the brining solution. An optimum concentration of $2N$ sodium chloride:$1.5N$ sodium hydroxide was reported. This is not in agreement with the author's finding for Type I strongly basic ion-exchange resins. The latter are relatively difficult to regenerate with caustic, and the Type II resins are not selective for chloride over hydroxide at equivalent fractions of 0.7 or more of chloride ion in solution.

On the other hand, it has been noted that after a single brine treatment has produced a peak organic concentration, tailing of organic material occurs. Caustic regeneration, however, again produces a high peak organic concentration. This cycling can sometimes be repeated up to 5–10 cycles until the organic concentration is quite low. No doubt this behavior is a reflection of the reorientation of polymer chains due to swelling of the gel when it is in the hydroxide form. This reorientation permits elution of the formerly entangled organic acids. Thus one can conceive of these large organic acids being bound by both normal ion-exchange forces and van der Waal's forces. The magnitude of the latter depends upon both the structure and size of the acids and of the ion-exchange polymer. With large acids of aromatic nature, strong binding forces exist between the foulant and the resin. Although brining can release some of the foulant, the slow diffusion rate of these large acids prevents ready removal. An order of magnitude estimate of the diffusion rates of these foulants can be made by comparison of the rates of loading, in a batch system, of organic dyes of modest molecular weights (Fig. 11). These sulfonic dyes were exchanged on the hydroxide forms of Resin A and its porous analog

Resin E. At a molecular weight level of 328, there are small differences in performance of the resins, either in rate or equilibrium loading. With the higher–molecular weight dye (mol wt 713) the rate of exchange by Resin A is nearly a magnitude higher than that of Resin E. Also, there appears to be a factor of four in relative loading of these resins. The rates of exchange are gel-diffusion controlled and ex-
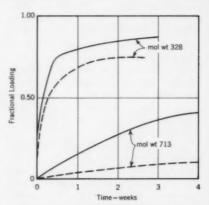

Fig. 11. Comparison of Exchange Rates of Resin A and Resin E With Sulfonic Dyes of Differing Molecular Weight

Solid curves represent performances of Resin A and dashed curves represent performance of its porous analog, Resin E.

tremely slow. From an analysis of these dye uptake data, we estimate that the capacity of Resin E for ions of 30Å or larger diameter is about 2.5 times that of the capacity of Resin A. Nevertheless, there is still a limited capacity of the porous resin which can accept the foulants' high molecular weight ($> 1,000$).

The fouling acids, believed to be humic in nature, can be produced by

the oxidation of naturally occurring materials such as cellulose, lignins, peat, or brown coal. Attempts at synthesizing these acids often involve oxidation of polyphenols. In nature, the decay process releases the humic acids. Proteinaceous matter from animal life is also believed to be involved. Raudnitz[8] has shown that leaves of the rhododendron contain a water-soluble, surface-active phosphoric ester of these acids. By careful hydrolysis he has produced a nitrogen and sulfur-free uniform humic acid. The acid was slightly soluble in boiling alcohol, insoluble in many organic solvents, but soluble in pyridine and aniline. A molecular weight of about 1,200, based on methoxyl content, was estimated. The acid reduced permanganate in alkaline solution. An infrared spectrum showed bands attributed to hydroxyl, oxygen, and carboxyl groups.

The surface-active nature of the foulant was also reported by Skold[9] who noted a stable heavy foam during regeneration of fouled resin.

Fuchs and Kohler[5] have characterized humic acid produced by the permanganate oxidation of brown coal by ultraviolet spectrum. Fuchs[4] has studied the humic acids and their effects on feedwater treatment. His attempts at complete removal by electrolyte flocculation were limited to 60 per cent removal. Alumina activated at 500°C was 99 per cent effective, however.

Van Beneden[10] presented evidence that some of the humic acids are complexed with iron and complete precipitation may be prevented by the protective action of colloids. He worked with peat extracts obtained from brooks. Although much of the older literature indicates that humic acids are insoluble in acids, van Beneden has noted the presence of humic acids at the outlet of feed beds where the pH was 3.9. This is in agreement with the finding of the solubility of the organic acid at low pH and with the study of Wilson.[11] Although van Beneden noted success with some extracts by flocculation in quite acid solution, other extracts did not flocculate in several hours. Also peak extracts containing high (humus) iron contents were most readily precipitated by acid addition. The extracts examined were about ten times more concentrated than the brook waters of Hautes-Fagnes which themselves appear yellow or brown in color. At these latter concentrations, flocculation with iron was rather difficult. This appears to be a general finding: precipitation techniques are only partially successful in dilute concentrations.

The acid-soluble fraction of humic acid and fulvic acid, was concentrated on carbon by Kononova.[6] This fraction could not be coagulated. A polymerized cyclic carbon structure with linear polymeric carbon chains was postulated.

Other Possible Sources of Fouling

Other sources of fouling substances, in addition to products of vegetation, have been suggested. They included bacterial products and cation-exchange resin degradation products. In some early investigation in the laboratory, resins were inoculated with various water microorganisms in solutions containing traces of various salts. The rate of attack was negligible.

The role of cation-exchange resin degradation in the problem of fouling has not been completely clarified. It is known that cation-exchange resins of the sulfonic type degrade in the presence of oxidizing agents.[2] This decross-

linking reaction is accelerated by many metallic catalysts, chiefly copper and iron. The product contains soluble sulfonic acids of low molecular weight. Experiments with the sulfonic acid "color throw" derived from the degradation of a 1 per cent DVB resin indicated that this material is highly selectively bound on strongly basic ion-exchange resins. Caustic regeneration was virtually ineffective. Similarly the extent of capacity loss was heavily dependent upon time of exposure, which suggests a gel diffusion–controlled process. In one instance an anion resin removed from a monobed was carefully separated from the cation resin and completely regenerated with salt to remove inorganic sulfate and other ions. Analysis of the resin revealed a sulfur content equivalent to 0.25 me/g. This could be attributed to polythionate, sulfur in the humic acid complex, or sulfonic acid. Although no evidence of sulfonic groups was noted in the infrared study of the eluted foulant, it is not possible to exclude cation degradation as a contributor to fouling. The presence of sulfonic groups in the degradation product would lead one to expect these products to be very tightly bound and not removable by conventional elution techniques.

Prevention of Fouling

Techniques recommended to reduce the extent of fouling are of three types: water treatment, proper resin selection, and resin cleanup.

Water treatment. Flocculation and slow sand filtration often permit removal of the organic foulants. Chlorination or oxidation may reduce the size of the foulant so that removal by back diffusion is possible. Each water supply must be evaluated individually to ascertain the most effective and eco-

nomical treatment. A possibility exists with some oxidants that the reaction will produce carboxylic groups on the materials which will make fouling more potent. Permanganate, which is often used in Europe for iron removal, may be disadvantageous in this instance.

Scavenger beds appear to be one of the most practical and effective means of preventing foulants from accumulating on the anion resin of a deionization unit. The chloride form of Resin E frequently regenerated with warm (110°–140°F) 5 per cent sodium chloride solution accumulates organic matter at a negligible rate. Carbon or alumina filters are useful but each location should be evaluated individually.

Resin selection. A porous resin such as Resin E, or F, its high-capacity analog, should be used for troublesome locations. They possess basicity nearly equivalent to that of Resin A, and, consequently, are of value for complete removal for weak acids as well as for reducing difficulties with organic fouling due to the high molecular weight organic bodies. The porous resins have a much higher capacity for these bodies than the conventional resins and they are also more efficiently regenerated.

Resin cleanup. Cleanup techniques involve frequent warm-brine treatments. The need to prevent the organic acids from penetrating far into the resin gel, where long regeneration periods will be necessary for removal, cannot be overemphasized. Warm caustic regeneration has been cited as being advantageous also.[1] Oxidative treatment, such as hypochlorite caustic techniques involving 0.5–2 per cent sodium hypochlorite in 1–2 per cent caustic, may be necessary when fouling has been permitted to build up. In some instances resins examined in this study were permitted to accumulate as

much as 5 per cent (by weight) of organic foulant. Under these conditions diffusion rates of even small inorganic anions were reduced appreciably by the influence of the large organic acid. In general, Type II resin cannot be treated with hypochlorite without excessive attack. Type I resins, including the porous analogs, are more resistant; and the latter withstand oxidative cleanup every 25 cycles for 300–500 cycles in the sugar industry.

Conclusion

Organic fouling of anion-exchange resins proceeds by the exchange of large organic acids, the humic acids produced by the decay of vegetable matter. These acids are commonly found in surface water supplies. Accumulation of the acids on resins limits the capacity of the resins for further exchange of the acids and inorganic ions present in raw water. Over a period of time, the acids diffuse into the resin gel and are tightly bound by van der Waal's forces in addition to the normal ion-exchange forces. The larger acids appear to be too tightly bound or to diffuse too slowly to be efficiently removed in the limited regeneration period, although elution with warm 5 per cent sodium chloride brine appears to remove significant amounts of the more readily released fraction. The most important technique in prevention of fouling and the attendant capacity and quality difficulties appear to be the limiting of organic accumulation on the resins. Thus, frequent brining and the use of scavenger beds appear to be most effective. Floc-

culation appears to be of limited value in many instances; however, each water supply must be examined individually before the optimum treatment can be deduced.

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History and Present Status of Chlorination Practice in Kansas

—Russell L. Culp—

*A paper presented on Apr. 21, 1960, at the Kansas Section Meeting,
Emporia, Kan., by Russell L. Culp, Acting Chief Engr., State Board
of Health, Topeka, Kan.*

DISINFECTION of water by chlorination had its beginnings in Kansas, as it did elsewhere in the United States, in about 1908. From 1908 to 1917 at least 30 Kansas towns started using hypochlorite solutions for water treatment. In 1917 equipment for the application of liquid chlorine became commercially available and the communities of Valley Falls and Coffeyville began using this new process. They were the first cities in Kansas, and among the first in the nation, to take advantage of the development. By 1919, Atchison, Leavenworth, Herington, Independence, Iola, Kansas City, Ottawa, Sabetha, Topeka, Wichita, and other cities had installed gas chlorination facilities, and additional public supplies were disinfected by means of hypochlorite solutions. The practice of chlorination spread rapidly, and by 1942, all 60 of the surface water supplies in the state and 40 of the ground water supplies were so treated. At that time there were, however, still 241 municipal ground water supplies that were not chlorinated.

Waterborne Outbreaks

In 1942 an epidemic of 3,000 cases of bacillary dysentery occurred in Newton, Kan., owing to contamination of the municipal well water supply after it had reached the distribution system.

As a result, Newton initiated continuous chlorination of its supply and many other Kansas communities using well water sources followed suit, in order to provide an additional barrier against the waterborne transmission of disease. Immediately following this incident the Kansas State Board of Health passed a resolution recommending the chlorination of all public water supplies in the state and the maintenance of adequate residuals in all sections of water distribution systems.

This occurrence of contamination of a well supply was not unique, but it did focus much attention on the problem in Kansas. Eliassen and Cummings [1] reported: "Distribution system contamination through cross connections and back siphonage were responsible for 42 per cent of the number of cases of disease attributable to public water supplies." They further concluded that "the use of ground water supplies without disinfection was the cause of more than half of these outbreaks," and that "disinfection should be mandatory for surface and ground water supplies in all water systems. . . ." The work of Eliassen and Cummings supplemented earlier studies covering the period 1920–36 by Gorman and Wolman,[2] who had found that "30 per cent of the outbreaks in the United States occurred where the water sup-

ply was from underground sources—largely shallow wells." Gorman and Wolman pointed out that "each new outbreak is a reverberation of the time-worn warning that relaxation in vigilance in safeguarding public water supplies takes its tragic toll of human lives and suffering, to say nothing of extensive economic losses to the community affected."

Early this year, following an outbreak of typhoid fever among consumers served by a municipal water supply system in New Hampshire, W. A. Hardenbergh editorialized in *Public Works* magazine as follows: [3]

The story is familiar to old-timers—a surface supply and no chlorination—but perhaps strange to those of us who feel strongly that every water supply should be adequately chlorinated, whatever its source; and that the absence of chlorination is a throwback to the darker ages of public health. There is available reliable equipment for applying chlorine and for regulating its application. The cost of insuring safety is small. The fourteen typhoid cases, including one death, represent a heavy price to pay for this neglect of established and proven public health practice in safeguarding water quality.

Chlorination and Quality

The reports cited above, which emphasized the need for chlorination treatment of all public water supplies, plus the cooperative efforts of local water department officials and State Board of Health engineers acting under the board's recommendations, produced substantial gains in the way of new chlorination installations and improved the bacteriologic quality of Kansas municipal water supplies from 1942 to 1956. During this period, 218 cities installed chlorination equipment, so that by 1956 all but 127 of the 445 city supplies were chlorinated.

During this same period the number of municipal water supplies failing to meet the USPHS Drinking Water Standard [4] for bacteriologic quality was reduced from 120 to 23. The direct correlation between improved chlorination practice and better water quality is a clear and dramatic illustration of the benefits to be obtained from the chlorination of well waters.

Two other experiences furnish information on the effects of chlorine residuals maintained in distribution systems. The first of these occurred in 1951 and 1952 when the State Board of Health asked the 70 cities in Kansas utilizing surface water supplies to maintain a minimum of 0.4 ppm free chlorine at all taps during the summer months. Ten cities replied that they were already maintaining residuals in excess of this amount. The other 60 cities were routinely maintaining a residual of 0.2 ppm, and all of them, so far as could be determined, increased the residual to at least 0.4 ppm. By comparing the bacteriologic records for these two summer seasons with those of the previous 5 years, it was found that the bacteriologic quality of the water, as determined by the coliform test, was improved in 55 of the cities, remained the same in four cities, and was poorer in one city. A number of cities have continued to maintain higher residuals in the system because of the favorable results obtained in these trials.

During initiation of chlorination of the well water supply at Sterling, Kan., in 1952, a series of special bacteriologic tests was made before and after chlorination was started. In addition to the standard coliform MPN determinations, total plate counts were made from a number of points on the distribution system. As might

be expected, there was a reduction in the number of positive tubes obtained in the coliform test following the start of chlorination, but there was an even more striking reduction in total plate counts, although the plate counts remained high for some time even after coliform organisms could no longer be detected. Initially, plate counts obtained from samples collected from the

throughout the system. The use of residual chlorine in the system to eliminate slime growths is well known and widely practiced in Kansas. It is particularly valuable in treating waters containing low concentrations of iron and manganese or high concentrations of sulfate to reduce corrosion, red water, and sulfide odors which would otherwise be caused by growths
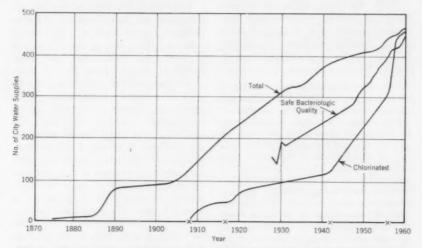
Fig. 1. Chlorination and Bacteriologic Quality in Kansas Municipal Water Supplies

The four upward inflections of the curve representing growth in the number of chlorinated supplies each represent a major event in the history of chlorination in Kansas. These are (in chronological order): the introduction of hypochlorite treatment, the development of gas chlorinators, the Newton epidemic, and the chlorination order issued by the State Board of Health

distribution system were considerably higher than for samples collected directly from the water supply wells. It was only after chlorination had been practiced for several weeks that it was possible to obtain satisfactory plate counts. Apparently, the only way to prevent the growth of certain organisms in the distribution system is to maintain adequate chlorine residuals

of iron bacteria or sulfate-reducing bacteria in the mains. Chlorine is also often introduced directly into wells to control growths of iron and sulfate-reducing bacteria, which have a tendency to clog well screens, eventually reducing the production of the wells.

During the 1952–57 drought, pressures in city distribution systems were often below normal. In several in-

stances, bacteriologically unsafe samples were collected in areas where pressure had been lost. Presumably, this contamination resulted from back-siphonage or cross connections. Comparisons showed clearly that satisfactory water quality was restored much more rapidly in cities practicing continuous chlorination treatment and maintaining chlorine residuals than in those that had to resort to emergency disinfection procedures. In the unchlorinated systems it was evident that accumulated pipeline deposits exerted a chlorine demand which seriously interfered with the emergency disinfection procedures. Few persons would question the fact that it is desirable or essential to maintain chlorine residuals in water systems under these and other emergency conditions, and it appears that this can be done much more effectively if chlorine residuals are maintained in the distribution system at all times.

Chlorine Residuals as Tracers

Chlorine residuals have also proved valuable as a warning device and tracer. In Washington, Kan., for example, although the original indication of contaminated water in the distribution system was given by a bacteriologic analysis, the measurement of chlorine residuals in the vicinity of a sample collection site made it possible to pinpoint the source of contamination quickly. A private well was found to be pumping water back through a check valve into the city water distribution system in the business district. In another city, similar investigation led to an open hatch cover in the roof of an elevated tank. Birds had entered in large numbers and contaminated the stored water. Chlorine residuals in the system had restricted the

spread of this contamination to a small area immediately adjacent to the tank.

Chlorine residuals are not a substitute for adequate plumbing codes, competent inspection, and elimination of cross connections and faulty plumbing, but they do provide an additional factor of safety. Cross connections and poor plumbing are installed at a much more rapid rate than even the best of inspection programs can control, and no public water system should be without the protection afforded by continuous chlorination treatment.

Chlorination Order

In 1956 the Kansas State Board of Health reviewed the progress that had been made pursuant to its resolution of 1942 recommending chlorination for all supplies. The board observed that there had been a great improvement in the bacteriologic quality of water supplies during the 14-year period, much of it unquestionably due to the new chlorination installations. Although 70 per cent of all municipal supplies were being chlorinated in 1956, there still were 127 unchlorinated well water supplies, 37 of which failed to meet the USPHS Drinking Water Standard[4] for bacteriological quality. It was known that many cities hesitated to initiate chlorination on their own, but that most, if not all, would respond to a specific requirement set by the board. Because additional progress seemed dependent upon further board action, in the interest of public health an order requiring the chlorination of all public water supplies in Kansas by Jan. 1, 1958, was passed on May 25, 1956.

Present Status

Since 1956, a total of 144 cities have installed facilities for continuous chlo-

rination, so that presently 462 of the 463 public water supplies in Kansas are chlorinated. During 1959, 442 of the 463 municipal water supplies met the USPHS Drinking Water Standard.

Figure 1 illustrates graphically the history of chlorination of water supplies in Kansas and its relationship to water quality. It shows the effects of four important landmarks in Kansas water chlorination practice: (1) the use of hypochlorite solutions beginning in 1908; (2) the development of compressed chlorine gas (liquid chlorine)

best in the United States. The universal acceptance of continuous chlorination treatment and the adoption of other protective measures and purification techniques have produced a remarkable reduction in waterborne disease and have greatly improved overall water quality. This record of accomplishment is something of which Kansas can well be proud. Nevertheless, 21 supplies in the state had unsatisfactory bacteriologic records in 1959, and that fact should not be overlooked. Although all of these systems were
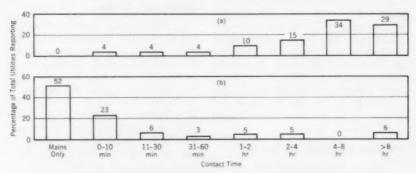
Fig. 2.   Chlorine Contact Time in Kansas Municipal Supplies

Parts a and b give contact times for surface and ground water supplies, respectively.

and the invention of equipment for its accurately controlled application to water in 1917; (3) the Newton waterborne epidemic of 1942; and (4) the State Board of Health's chlorination order of 1956. Each of these four landmarks has been followed by a period of accelerated adoption of chlorination by cities with resulting improvements in water safety.

Chlorination practices in Kansas have shown steady and sometimes rapid progress since 1908 and, at the present time, probably rank among the

small ones serving less than 0.5 per cent of the population using city water, they are evidence of a lack of trained or skilled operation owing to the rapid turnover in water utility managers and operators in these small towns. Now that all cities do have chlorination equipment, however, more emphasis can be given to its proper operation and to the training of new personnel. Thus, it is now possible to look forward to a fifth landmark in Kansas chlorination practice—the satisfactory operation of all chlorination equipment

—which should make it possible for all public water supplies in the state to meet consistently the USPHS standard. This is something the water-using public expects—or, more accurately, takes for granted—and there appears to be no reason why it should not be accomplished.

Statewide Survey

To obtain information which might assist in working toward the goal set forth above, a statewide survey of present chlorination practices in Kansas was made during 1960. Response to a four-page questionnaire, which was sent to each city, was excellent, and some interesting facts were revealed. From state records it was known that 462 of the 463 municipal water supplies in the state were chlorinated. The rest of the data given below were compiled from the questionnaires returned by 259 cities, representing 55 per cent of all municipal supplies in Kansas.

Equipment and Controls

A breakdown of chlorinator installations by type of feeder revealed that 49 per cent of the cities use solution feed pumps, 37 per cent gas chlorinators, 13 per cent tablet feeders, and 1 per cent drip feeders or other homemade equipment.

There is a nearly equal use of three methods for controlling chlorination equipment: 36 per cent are manually controlled, 31 per cent are semiautomatic, and 33 per cent have fully automatic proportional control.

Points of Application

In well water systems having no treatment other than chlorination, 58 per cent apply the chlorine solution into the pump discharge line, 35 per cent into the well, and 7 per cent ahead of a ground level storage reservoir. In well water treatment plants, principally lime-soda softening or iron removal plants, chlorine is first applied to the raw water in 55 per cent of the plants, to the filter influent in 17 per cent, and to the filter effluent in 28 per cent. In surface water treatment plants, 69 per cent prechlorinate the raw water, 6 per cent apply the first chlorine dose in the final settling basin, 10 per cent apply chlorine to the filter influent, and 15 per cent to the filter effluent.

Of the surface water plants, 62 per cent normally apply chlorine at two or more points in the treatment process: 17 per cent apply it at three points (raw water, filter influent, and filter effluent) and 45 per cent dose at two points. There is some variation in the twin points of application used: 31 per cent divide the chlorine between the raw water and the filter effluent, 8 per cent between the raw water and the filter influent, and 6 per cent between the filter influent and effluent. There are also differences in practice where only a single point of treatment is used. Of the total 38 per cent utilizing a single dose, 19 per cent rely on prechlorination only, 4 per cent on application to filter influent only, and 15 per cent do not add chlorine until all other treatment is completed or following filtration. Of the surface water plants, 27 per cent routinely practice free residual chlorination and 73 per cent do not. The survey also revealed that 8 per cent of the plants reporting add ammonia or ammonium compounds, whereas 92 per cent do not.

Contact Time

Chlorine contact time is particularly important in the processing of water from surface sources. The time required for disinfection depends upon chlorine concentration, pH, temperature, turbidity, and other factors. The time available in Kansas surface water plants is given in Fig. 2a. It should be noted that 63 per cent provide more than 4 hr of contact. This is, of course, one of the advantages of prechlorination, in that it utilizes the settling basins for increasing chlorine contact. Another 25 per cent provide 1–4-hr contact. Presumably, these plants apply chlorine to the final settling basin or filter, which is probably sufficient for low-turbidity water. The other 12 per cent provide up to a 60-min contact in the treated-water reservoir or clear well. This is a questionable practice in the disinfection of surface waters, especially for that 4 per cent with less than 10-min contact.

As might be expected, the chlorine contact time provided for well water supplies (Fig. 2b) is much less than for surface waters. The principal reason for this is that special structures or repumping may be necessary if long contact is to be obtained. Figure 2b shows that 52 per cent of all well water supplies surveyed provide only that contact which takes place in the water distribution system. Another 23 per cent afford less than 10 min of contact, an additional 9 per cent provide 11–60 min, and only 16 per cent have contact periods in excess of 1 hr. There is room for considerable future improvement in this regard.

Chlorine Residuals

Figure 3 shows some of the wide variations prevalent in present practice

with regard to chlorine residuals. Figure 3a indicates the chlorine residuals maintained at plant taps. These range from 0.2 ppm to more than 2.0 ppm. The question of what chlorine residuals are maintained at taps most distant from the water plant is a difficult one to answer because of the different interpretations that could be made, but the summary of replies given in Fig. 3b should yield some general information at least. It should be noted that 6 per cent of the cities do not have
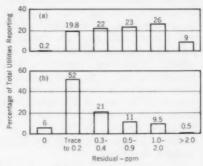
Fig. 3. **Chlorine Residuals Maintained in Kansas Municipal Water Systems**

Parts a and b give data on chlorine residuals maintained at plant taps and at taps most distant from the treatment plant, respectively.

any residual chlorine at remote taps, and that 52 per cent have only a trace to 0.2 ppm. The nature of the chlorine residual maintained in city water systems varies as follows: 49 per cent of the systems carry all combined chlorine, 23 per cent part free and part combined, and 28 per cent all free chlorine.

The orthotolidine color comparator is by far the most common method employed for testing chlorine residuals, with 92 per cent using it. Only 6 per

cent use amperometric titration, and 2 per cent have continuous chlorine residual recorders.

The frequency of chlorine residual tests is Kansas plants is greatly affected by the large number of well water supplies in the state. In most of these supplies there are few or no variations in chlorine demand, water quality, temperature, or other factors which would require frequent changes in the rate of chlorine application. Thus a minimum of tests often suffices. One operator reported that he checked the chlorine residual twice a year, once in the summer and once in the winter. This is a little extreme, and much can be said for making daily checks, as 26 per cent of the operators reporting do. Nevertheless, 54 per cent make chlorine tests less than once daily. The other 20 per cent follow more rigorous testing schedules which vary from hourly to twice daily.

Nondisinfection Benefits

Chlorination serves many purposes other than disinfection, some of which were observed and reported by the systems surveyed. For example, 72 per cent of treatment plants for the removal of iron, manganese, or sulfides use chlorine as an aid in the removal. Half of the well and surface water filter plants reporting receive some benefit from prechlorination for control of algae in settling basins; 20 per cent of the plants with aerators use chlorine to restrict algae growths in this part of the plant; and 22 per cent consider chlorine to be an aid to flocculation. Some of the added benefits apply to both surface and ground water supplies : 30 per cent reported benefits from chlorination for control of tastes and odors; 19 per cent have found chlorine useful for control of iron bacteria,

sulfate-splitting bacteria, or other slime organisms in mains; and 3 per cent have located cross connections or other sources of contamination by observing reduced chlorine residuals in the vicinity. In 19 per cent of all well water supplies reporting, chlorine is introduced directly into the well to control growths of iron and sulfate-reducing bacteria and other slime organisms that would otherwise clog well screens.

Chlorination Difficulties

The last question in the survey had to do with the problems experienced with chlorination treatment. Of the cities replying, 75 per cent reported no problems with chlorination treatment or equipment. Difficulties with feed equipment were described by 10 per cent. More than two-thirds of these problems related to the plugging of hypochlorinators or feed lines by calcium deposits, the plugging of tablet feeders or lines by calcium deposits, or the frequent cleaning required to keep gas chlorinators operating. There were also a few scattered complaints about the loss of prime by feed pumps, the delays encountered in obtaining repair parts or service, and deposits in chlorine evaporators. Another 15 per cent had experienced some problems with the chlorination treatment process. The majority of these problems had to do with the difficulty in maintaining chlorine residuals in parts of the distribution system distant from the plant, or in dead-end mains. Some public opposition to chlorination was reported in six cities, but nine cities gave unsolicited praise of chlorination in reply to the request for a listing of problems. There were a few scattered reports of other problems: (1) precipitation of iron from well water after chlorine is introduced; (2) con-

sumer complaints of taste and odor following installation of new service lines; (3) difficulty in maintaining free chlorine residual because of old accumulated deposits in mains; (4) frequent adjustments of chlorine feed rate necessitated by the fluctuating chlorine demand of the raw water. One operator complained because his plant was not designed to provide chlorine contact time at a low pH ahead of lime-soda softening.

Conclusion

Four events have had a significant effect on chlorination practice in Kansas: (1) the discovery and use of hypochlorite solutions in 1908; (2) the development of compressed chlorine gas and the development in 1917 of equipment to feed this material; (3) the Newton waterborne epidemic of 1942; and (4) the Kansas State Board of Health's chlorination order

of 1956. Each of these events has been followed by an upsurge in installation of chlorinators, and in each instance there has been a corresponding improvement in the bacteriologic quality of public water supplies.

At present, with only one exception, all municipal water supplies in Kansas are chlorinated, and it is expected that with increased emphasis on proper operation it will soon be possible to realize even further gains in water safety.

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Algae and Other Interference Organisms in the Waters of the South Central United States

C. Mervin Palmer

A paper presented on Oct. 19, 1959, at the joint meeting of the Southwest and Alabama-Mississippi sections, New Orleans, La., by C. Mervin Palmer, Biologist, Robert A. Taft San. Eng. Center, USPHS, Cincinnati, Ohio.

THE microscopic organisms found in all surface water have an important influence on its quality and its many and various uses. They affect its physical and chemical characteristics; they serve as food for fish and other large aquatic animals; they influence its palatability and complicate and interfere with its treatment or preparation for household and industrial use. On the other hand, they serve in the self-purification of polluted streams and are put to use in stabilization ponds for the treatment of sewage.

Many groups of aquatic plants and animals comprise the population of microorganisms that are capable of spending all or much of their life cycle in the water. These include algae, bacteria, fungi, protozoa, crustaceans, nematodes, and numerous others. Those that have been found to cause problems in the treatment and use of the water are known as the interference, or nuisance, organisms. Many kinds of algae are particularly well known for their tendency to cause such difficulties.

South-Central States

The six south-central states of Alabama, Arkansas, Louisiana, Mississippi, Oklahoma, and Texas, have their share of problems caused by interference organisms. This account, the fourth of a series [1-3] dealing with interference organisms in various regions of the United States, represents a summary of reports describing conditions and problems in the south-central states and the methods used to combat them.

There are 25 drainage basins in the 500,000-sq mi area under consideration, including the lower Mississippi, Tennessee, Tombigbee, lower Arkansas, Sabine, and the Rio Grande–Pecos river basins. They cover an area with an average annual rainfall ranging from a scant 5 in. in the West to more than 60 in. in the Southeast. In western Texas, 26,000 sq mi contribute little or no surface runoff to the streams,[4] whereas in Alabama 50 per cent of the rainfall reappears as runoff to the streams, the other 50 per cent being lost principally through evaporation.[5]

Most of the streams of Alabama, Arkansas, Louisiana, and Mississippi, as well as those in the eastern half of Oklahoma and Texas, contain soft waters of low mineral content. The harder, more highly mineralized surface waters, showing high concentrations of calcium, sulfate, and chloride, are typified by that of the Red River.[6] The Arkansas River is unsuitable for domestic and most industrial uses because

of its high mineral content. Little Rock, Ark., was the last municipality between the mouth of the river and the Colorado State Line to abandon the river as a public supply.[7]

In the eastern half of the six-state area there are many natural and artificial lakes and ponds scattered throughout the extensive coastal plain. The reservoirs and lakes are used for a number of purposes, such as sources of public water supplies, recreation, fishing, water power, flood control, and water conservation. Many of them are designed to serve more than one purpose. Alabama, in 1958, had nineteen large impoundments covering a total of 285,325 acres and an additional 170,000 acres under construction or in advance-planning stages.[8] Some of the states also have large numbers of minor impoundments of 100 acres or less, there being more than 10,000 of them in Alabama.[9] Mississippi has more than 50,000, classified as farm ponds.[10]

Typical chemical analyses of an impounded water and of both a soft-water and a hard-water stream are given in Table 1.[8] The waters of Alabama are comparatively soft and are often both high in iron and corrosive to distribution systems unless given corrective treatment.[11] Louisiana has an abundance of ground water as well as many sources of surface waters, but the latter requires considerable treatment for use for domestic supply. The surface water has a relatively high temperature and, therefore, has been considered impracticable for use for industrial cooling. The majority of domestic water supplies in the state are ground waters, except in the region along and west of the lower Mississippi River and at the cities of Monroe and Shreveport in the north.[12]

Types of Algal Growth

The temperature, turbidity, pH, color, and concentration of nutrients in the water are some of the major factors that are responsible for distinctive features of the algal growth in the south-central states. This growth is distinc-

TABLE 1

Chemical Analyses of Three Surface Waters in the South-Central United States

Item	Fort Smith Lake, Ark.	Red River at Denison Dam Tex.-Okla.	Comite River, Comite, La.
Silica (SiO)—*ppm*	4.3	12	16
Iron (Fe)—*ppm*	0.20	0.02	0.58
Calcium (Ca)—*ppm*	12	94	3.7
Magnesium (Mg)—*ppm*	2.0	22	1.3
Sodium (Na) & Potassium (K)—*ppm*	2.6	150	6.8
Carbonate (CO₃)—*ppm*	0	0	0
Bicarbonate (HCO₃)—*ppm*	28	135	16
Sulfate (SO₄)—*ppm*	12	188	3.7
Chloride (Cl)—*ppm*	4.5	242	6.2
Fluoride (F)—*ppm*	0.0	0.04	0.7
Nitrate (NO₃)—*ppm*	1.4	2.2	1.7
Dissolved solids—*ppm*	53	829	75
Total hardness as CaCO₃—*ppm*	38	325	15
Specific conductance*—*μmho*	84	1300	59
pH	7.2	7.9	7.0

* At 25°C.

tive in a number of ways. Comparatively few kinds of algae are to be found during the hot summer months. Those that do survive are mostly a few genera of blue-green algae and certain filamentous green algae. Diatoms are not often abundant, whereas desmids form a significant part of the algal popula-

tion. The high temperature and other factors often combine to cause algae both to develop quickly and to be destroyed quickly. Plankton algae in flowing streams of this region are reported to be few in number, owing to adverse chemical and physical conditions of the water. Ponds in some parts of the area are almost devoid of algae, owing to a lack of nutrients, high turbidity, the dark color of the water, or competition with dense growths of higher aquatic plants. In other ponds, however, the algal growth may be very luxuriant, particularly during the spring and fall.

Desmids are abundant in a large part of Mississippi, Louisiana, the granitic regions of Oklahoma, and in eastern Texas. They tend to be littoral rather than planktonic. The area appears to be as rich in desmids as any district in the United States. Several hundred species have already been reported, some being listed for the first time in the country.[13] Some of the most common desmids in the south-central states are *Closterium, Cosmarium, Desmidium, Euastrum, Gymnozyga, Hyalotheca, Micrasterias, Netrium, Staurastrum,* and *Xanthidium.*[13-14] The presence of water lilies and bladderwort serves as an excellent indication of the presence of desmids. The waters tend to be brownish in color, low in nutrients, and neutral to acid in reaction. Mixed with the desmids and common in other parts of Oklahoma are the filamentous green algae belonging to the families Zygnemataceae and Oedogoniaceae.[15] In Louisiana, some filamentous green algae, such as *Spirogyra, Pithophora,* and *Hydrodictyon,* often form such extensive mats in rice fields that they injure the crop.[16] In Alabama, *Pithophora* often becomes the predominant growth in fish ponds dur-

ing the summer. It interferes with high fish production by reducing the growth of phytoplankton required directly or indirectly as fish food. The *Pithophora* also interferes with the harvesting of fish by forming a heavy blanket of growth that floats on the pond. The growth can also promote overpopulation of bluegills by interfering with bass predation.[17]

The alkaline marshes in Mississippi and Louisiana, as well as several lakes and smaller farm ponds on the blackland terrain of eastern Texas and the waters of western Texas, are some of the areas in which blue-green algae predominate.[18] In addition, waters which are low in calcium but high in nitrogen are also likely to develop large numbers of blue-green algae, particularly *Anacystis (Microcystis), Oscillatoria,* and *Aphanizomenon.*[19] In rice fields of southern Louisiana the water is strongly alkaline and blue-green algae predominate, including the genera *Anabaena Aulosira, Gloeotrichia,* and *Nostoc.*[16]

Many of the larger impoundments throughout the six-state area contain plankton flora, particularly during the spring and fall, composed principally of many kinds of unicellular and colonial green algae and flagellates. Lake Hefner, Okla., was found to have a population primarily of green algae and flagellates with the highest numbers occurring in the summer. The dominant genera included *Ankistrodesmus, Coelastrum, Phacus, Phacotus, Pediastrum,* and *Sphaerocystis.*[20] *Pediastrum* also was reported to be predominant in the impoundment at Decatur, Ala.[21]

In many Texas waters microscopic life reaches a peak in spring and early summer with a drop in late summer

and a small rise in the autumn. Spring and fall maximums of the diatoms are more marked than those of the green and blue-green algae. The peak of diatoms usually occurs in April, with another in late fall. Green algae are most abundant in June and blue-green algae in early fall. Protozoa and flagellate algae are common in early spring, whereas crustaceans are more abundant in early summer.[22]

Some of the larger impoundments in central Texas have reported the dominance and even blooms of diatoms, including *Melosira, Synedra, Tabellaria, Fragilaria,* and *Asterionella.*[19] Lake Overholser, Okla., contained more diatoms than other algae, with a relatively uniform population throughout the year, two of the dominant genera being *Cyclotella* and *Melosira.*[20] In Pickwick Reservoir in Alabama, diatoms were found to predominate during the early part of the year, followed by green and blue-green algae in the summer.[21] When silicates are already present from the colloidal clay of the surrounding soil, an increase in the supply of phosphate in the water often results in a diatom bloom. If the water is turbid, it is likely to be a growth of *Melosira* or *Synedra,* and in clear water it may be *Asterionella, Fragilaria,* or *Tabellaria.*[19] Diatoms generally thrive best in cool water having a temperature in the range 40°–65°F. Therefore, in many of the smaller or shallow lakes and reservoirs of the south-central states, the diatoms are kept to small numbers during the summer because of the high temperature of the water.

River Plankton

In contrast to most of the ponds, lakes, and reservoirs, the rivers of the six states appear to have a predomi-

nance of diatoms. The most common genera are *Cyclotella, Melosira,* and *Synedra.* Other diatoms that have been abundant are *Amphiprora, Asterionella, Cocconeis, Gomphonema, Navicula, Nitzschia, Pinnularia,* and *Stephanodiscus.*[23]

Plankton analyses of water samples from four rivers in the south-central states have been made over a period of 2 years as a part of the USPHS water quality basic-data program.[23] The records to date make it evident that the number of phytoplankton can vary from zero to more than 100,000 organisms per milliliter (clump count).

TABLE 2

Plankton Algae From Four Rivers of the South-Central States

Organism	Organisms per milliliter	
	Avg	Max.
Total algae	4190	103,370
Centric diatoms	2210	49,780
Pennate diatoms	870	12,240
Blue-green algae	450	20,830
Green algae	430	16,620
All other algae	230	7,020

The average number for the 2-year period was approximately 4,000 organisms per milliliter. Table 2 gives the average and maximum numbers of algae and the number per algal group for the 2-year period. The samples considered came from nine collecting stations located on the Rio Grande and the Arkansas, Mississippi, and Red rivers. The Arkansas River averages over 3,000 per milliliter and the Red River averages about 500 per milliliter.

Non-diatom algae that tend to be abundant in the rivers of the area include the blue-green genera *Oscilla-*

toria, Phormidium, Anacystis, Agmenellum (Merismopedia), Anabaena, and *Spirulina,* the flagellate *Chlamydomonas,* and the green algae *Selenastrum, Scenedesmus, Actinastrum, Closterium, Chlorella, Oocystis, Ankistrodesmus, Gloeocystis,* and *Dictyosphaerium.*

It is evident that aquatic microorganisms are significant, in both kinds and numbers, in the streams as well as in the lakes and ponds of the area. In spite of high summer water temperatures, high turbidity, and low concentrations of nutrients, algae and other aquatic microorganisms are capable of causing problems in the treatment and use of water. These problems include tastes and odors, and coloration of the water, clogging of sand filters, infestation of distribution systems, decreasing the effectiveness of chemical treatment, and interference with the various uses of impoundments, ponds, and streams.

Tastes and Odors

More than 70 water treatment plants for the cities and towns in the six-state area have reported the occurrence of tastes and odors and the use of treatment to combat them. Hopkins, as early as 1926, emphasized that, although the taste may not be directly detrimental to public health, its presence in the finished water causes the public to lose confidence in the water utility.[24]

The sources of supply in which tastes and odors due to organisms occur are streams, lakes, impoundments, canals, or other surface waters. Until recently, the role of industrial wastes as a significant cause of tastes and odors in public water supplies has not been emphasized. Decomposition products from organic matter, including dead microorganisms, the metabolic products of algae, and the products of actino-

mycete growth have all been listed as the most significant causes of the tastes and odors in the south-central states.

Sometimes the large quantity of decomposing algal material gives rise to tastes and odors. The algae involved are likely to be a limited number of genera of blue-green algae during the summer, and a mixed growth of many kinds of green algae and flagellates during the spring and fall. Sometimes certain kinds of algae, even in small or moderate numbers, are responsible for tastes and odors. Some of the algae of the area reported to be responsible for the more specific odors and tastes are the diatoms *Melosira* and *Synedra* in Alabama [21] and *Synura* and *Euglena* in Texas.[25]

Cohen stated that the sources of tastes and odors in surface waters in Texas are decomposition products of dead algae.[26] In Alabama the foul odors in reservoirs were considered to be due to the decomposition of land flora which had been submerged, or to decay of catchment and littoral vegetation which was being carried into the reservoirs.[5] Reeves stated that algae such as *Anabaena, Asterionella,* and *Rivularia,* common in Texas, caused odors in water by their natural life processes, and that with certain plants decay or heating merely intensified the natural tastes and odors, whereas with others the odors were noticeable only during decay.[27]

In Lake Bachman near Dallas, the increase and decrease of the water odor was found to correspond exactly with the increase and decrease of a single species of the algae.[28] The most common growth at Dallas was a filamentous blue-green form related to *Anabaena,* and capable of producing moldy or musty odor or taste.[29] Silvey reports that undesirable odors in water are due

to the byproducts of actinomycetes. These bacteria are reported to act on dead and living algae and on other organic substances in the water.[30]

Billings [31] stated that the type of taste and its peculiarities of formation are different in Texas from those in the eastern, western, and northeastern sections of the United States. The most prevalent odors in this six-state area are reported to be the fishy, woody,[33] rotting-wood or lake bottom, and putrid or vile odors.[34] These are most common in the late summer and fall but are also frequent in the spring. These periods of intense odor correspond to the periods of most active algal growth in the reservoirs, lakes, and ponds. Some of the communities that have reported severe taste and odor during the spring are Shreveport, La.,[35] with a woody-fishy odor; Guntersville and Decatur, Ala.,[36] where the odor was variously described as musty, earthy, and grassy; San Benito,[37] Tyler,[38] and Waco,[39] Tex., with a rotting-wood taste and odor. Most of these communities also have had a recurrence of the taste-and-odor problem in the fall. Dallas and other cities and towns usually experience the most serious odor problems in the fall.[39]

The summer temperature of the water may reach 84°F or higher. Lake Dallas reached 98°F in August 1946.[40] Summer stagnation may occur especially near the bottom of impoundments. In the fall turnover, the warmer water from the bottom rises and may bring many of the taste and odor compounds to the surface.[18]

An increase in the intensity of tastes and odors may occur during periods of decreased rainfall, especially in the summer. As the water level of the reservoir falls, the algae growing near the shallow beaches are killed, and, owing to the high water temperature, decomposition occurs in such a manner as to produce tastes and odors.[41] On the other hand, heavy summer rains may also bring about a taste and odor problem. Microscopic biological matter washes into the reservoir with every sizable rain. Ten days of hot weather are sufficient to kill these organisms and produce an objectionable odor.[42]

Communities that obtain their supply from rivers and streams also encounter taste-and-odor difficulties. Activated carbon has been used for odor control at Jackson, Miss., using Pearl River; Meridian, Miss., using Long Creek; Brownsville, and El Paso, Tex., using the Rio Grande; Port Arthur, Tex., using Neches River; Sheffield, Ala., using the Tennessee River; Florence, Ala., using Cypress Creek; Archadelphia and Malvien, Ark., using Ouachita River; Jefferson Parish and Thibodeux, La., using the Mississippi River; and Houma, La., using the Intercoastal Canal.[43, 44] At Marrero, La., which received its supply from the Mississippi River, the taste-and-odor problems were practically absent until about 1944, but for the next 5 years, they had a steady increase with a strong medicinal, tarry taste and odor, producing a threshold odor up to 40.[45]

Records of plankton analyses of five rivers in the six-state area indicate that, although the algae are more abundant in the spring and fall, the number of microorganisms is also large during the summer and winter.[23] Weather and stream conditions might easily make tastes and odors as well as other problems due to plankton possible during any season.

Treatment Plant Problems

Interference organisms bring about problems in the water treatment plants

by shortening filter runs, clogging intake screens, forming slimy layers of growth on the walls of filters, settling basins, intake pipes, or on the rough surfaces of aerators, and increasing sludge deposits in settling basins. The organisms also change the pH, alkalinity, carbon dioxide and oxygen content, color, and turbidity of the water.[29] Tastes and odors may develop in the treatment plant even when they are not evident in the raw water.

Growths on the basin and filter walls are not only unsightly but they may also slough off to increase the organic material that has to be dealt with during sedimentation and filtration.[47] When algae change the physical and chemical characteristics of the water, frequent adjustments in the treatment process are required to prevent poor coagulation and insufficient settling and to compensate for changes in the chlorine demand, pH, and oxygen content of the treated water.

At Waco, Tex., when some of the filters were out of service for 2 days or more, the organic debris from dead algae and other microorganisms decomposed in the water and sand of the idle filters. When these filters were put into operation again, severe taste-and-odor troubles developed and spread through the distribution system, resulting in serious protests from the consumers. When the idle filters were washed thoroughly before being put back into use, no taste-and-odor trouble developed.[47]

Port Arthur, Tex., reported short filter runs caused in part by the presence of algae in the sand. The algae originated from the raw-water reservoirs or from growths on the settling basins and filters.[48] At Fort Worth, Tex., it was reported that after a "heavy run" of algae, the filters some-times developed cracks readily. There also were times when head losses increased quite rapidly without a corresponding increase in the turbidity of the applied water. This was considered to be due to diatoms and other algae, as was later verified by microscopic examination of the water.[49]

Algae have been present in the Rio Grande in such high numbers that filter runs of less than an hour were experienced at the Loredo, Tex., plant. It then required almost as much water to wash the filter as the filter would produce during the run. The long, slender diatoms are especially effective in producing short filter runs. They seem to form an almost impervious mat on or in the top layer of sand, causing a rapid increase in head loss.[50]

Lufkin, Tex., used well water as a source of supply and, after aeration in a coke bed, stored it in open concrete tanks from which it flowed into the distribution system. Operation of this system was hampered during the summer months by heavy growths of the green alga, *Spirogyra*. Growth became so heavy and thick that it necessitated regular weekly emptying and cleaning of the tanks. More than 1.5 ppm chlorine was required to kill the algae within 6 hr.[51]

At Mexia, Tex., the water supply was a combined limestone spring and deep well supply which was mixed in a large settling basin. *Oscillatoria* and *"Protococcus"* developed in large numbers. The water turned green and had a strong fishy odor. It required 3 ppm chlorine to inhibit the algae in the basin.[32]

Operational problems due to interference organisms, as experienced in small water plants of Arkansas and nearby states, have been summarized by McGoodwin.[52] He states that inade-

quate intake screens allow fish, debris, and other material to enter the pumps or flow line. Transmission lines carrying raw water often become coated with algae and algal slime causing loss of capacity, increased pumping costs, and taste-and-odor problems. These conditions are especially acute in low-velocity lines. Algal growth in raw-water storage tanks is sometimes serious, especially if subjected to sun and wind when tanks are shallow. The pretreatment units, and in many plants the treatment units, are uncovered, thus stimulating growth of algae both in the water and on the basin walls.

Intake lines carrying raw water from the reservoir to the treatment plant are subject to the development of an organic slime layer on the inside pipe walls. At Little Rock, Ark., a new intake line, 39 mi long, 39 in. ID, and constructed of steel cylinder reinforced-concrete pipe, was put into service. After 1 year the capacity of one section of the line had been reduced from more than 25 mgd to approximately 20 mgd. Inspection of the line showed a thin, dark organic slime growth on the walls. Although the layer was loosely adherent it was capable of increasing the friction to the flowing water. A laboratory study revealed that three types of bacteria predominated in the slime: large capsulated forms, small capsulated forms, and the iron bacteria known as *Crenothrix*.[55] These organisms have also caused trouble in the transmission line at Longview, Tex.

Infestations in Distribution Systems

Treated-water reservoirs, cisterns, and distribution lines are subject to infestation with various types of organisms. Chironomid worms, particularly the blood-red ones, are large enough to be visible. These are larvae, which in the adult stage emerge as midge flies. Other infestations may involve crustaceans, nematodes, iron and sulfur bacteria, and algae.

Chironomus infestations have occurred in Louisiana at Church Point, New Roads, Rayne, and the Louisiana Experiment Station. At Church Point, the red worms were first noticed in bath water. They were suspected of coming from the reservoir and several control measures were carried out, but permanent control was found to be difficult.[54]

Houston reported difficulty in some areas of the city because of algae growths in the mains. These growths would collect in the small meters and on the sides of the pipe. Systematic flushing from fire hydrants brought good results.[55] Algae were present in the distribution system occasionally at Witchita Falls, Tex., when, during periods of high demand, much water was bypassed around the filters.[56] Pretreatment with chlorine-ammonia was used to serve both as an algicide and an effective sterilizing agent.

Although deep well water is free of algae when first drawn, some communities find it necessary to store this water in reservoirs until needed. If these reservoirs are open, algae will develop, often very rapidly and abundantly. At El Paso, it was necessary to clean the open reservoirs at least twice during the summer because of the algae. The reservoir was later covered. It was concluded that, although the costs of covering of the storage reservoirs might have been high, the results obtained are well worth the expense.[57]

Interference Organisms in Impoundments

Aquatic weeds continually threaten to take over the more shallow portions of reservoirs and ponds. At Lake

Austin, Tex., *Myriophyllum* has been abundant.[58] This plant tends to develop in lakes which are held at a constant level. At Lake Crook, Paris, Tex., the American lotus gave considerable trouble, at one time covering 300 acres. It was probably introduced by wild ducks and geese. To control it by cutting, it was necessary to cut the plants 12–14 in. below the water surface and before the blooming stage. Three years of cutting were required to bring the infestation under control.[59] Lake Waco, Tex., has reported the presence of cattails, sawgrass, willows, and other higher plants, the dead remains of which could serve as the source of noticeable tastes and odors when acted upon by bacteria.[19] Reeves stated that the larger forms in reservoirs which affect potable water are chiefly water hyacinth, pond lily, cattail, lotus, duckweed, floating evening primrose, water milfoils, sedges, and water fern.[27] Shreveport, La., was troubled with such underwater growths as weeds and lily pads and instituted a cleanup program to remove them from the reservoir.[35] Tyler, Tex., with a 177-acre lake as its source of water supply, reported a great deal of vegetation, such as flags and lily pads, growing along the very shallow shoreline.[38]

Obnoxious weeds and algae are rapidly becoming pests in lakes and ponds in most areas of Alabama.[60] Water hyacinth is considered an obnoxious aquatic plant, especially in the Mobile Delta section of southern Alabama, where it has interfered with recreational uses of the water. Before the effectiveness of 2,4-D was discovered, dynamite and conveyers were used in fruitless attempts to control the growth.[61]

In the ponds and backwaters of reservoirs in Alabama, large bryozoan colonies are found. They may be brown-coated masses of stiff jelly attached to sticks and stumps. The masses frequently trap gas bubbles and float on the water. They give off slime when touched and serve as habitats for the larvae of gnats and midges. During the late summer, they may become unsightly when they grow to be a yard or more in diameter. In the fall, they are produced in such numbers that they may form windrows along the margins of the ponds.[62]

In the reservoirs many kinds of algae are responsible for unsightly blooms, surface mats, dense attached growths in the shallow areas, and windrows along the shoreline. They are the primary source of tastes and odors which may be objectionable both to people living near the reservoir and to the users of the water. The algae that tend to form blooms in reservoirs in the six-state area include the blue-green algae *Anabaena, Oscillatoria, Anacystis (Microcystis)*, and *Aphanizomenon*; the green algae *Scenedesmus* and *Pediastrum*; the flagellates *Volvox, Chlamydomonas*, and *Pandorina*; and the diatoms *Melosira, Synedra, Tabellaria, Fragilaria, Asterionella, Navicula*, and *Pinnularia*. Blooms of *Melosira* have occurred in lakes Amirillo, Bridgeport, Caddo, Dallas, Eagle, Waco, and Worth in Texas,[19] and at Wilson Dam and Decatur, Ala.[21] Blooms of *Anabaena* have been observed in lakes Hamlin, Ranger, Possan, Kingdom, and Wichita, Tex.,[19] and in various reservoirs in Louisiana.[16]

Attached algae that are troublesome in reservoirs include certain filamentous green algae. *Dichotomosiphon* developed in a reservoir at Bastrop, La., in July.[16] *Pithophora* grows in profusion in Alabama during the warmer months of the year. It develops first on the bottom in the shallow areas and later forms floating mats.[17] *Cladophora* is

one of the sessile algae attached to the riprap and periphery of Lake Hefner at Oklahoma City, Okla. It was estimated that the sessile algae of Lake Hefner contributed three times as much organic matter by weight for odor production as that contributed by plankton.[37]

Toxic Algae

During June and July of 1953, in a period of a little more than 6 weeks, thousands of fish died in Lake Austin, Tex. Most of the fish were gizzard shad which feed on plankton. At this time a bloom of one of the red-brown flagellate algae was present in such large numbers as to give the water a blood-red color. The alga was classified as one of the dinoflagellates and was identified as either *Gymnodinium* or the closely related *Glenodinium*. Organisms responsible for the toxic "red tides" in the Gulf of Mexico and other marine areas are also dinoflagellates. This appears to have been the first time on record that a fresh-water representative of the group showed evidence of being toxic. Scientists who studied the case concluded that the gizzard shad died of the toxic effect of the dinoflagellates. They found that the alga grew well only under specific conditions of light, temperature, and nutrients and that the right combination of these conditions was not likely to occur very often.[63]

A number of blue-green algae are known to be extremely toxic sporadically to many kinds of water birds, fish, and land animals that drink water containing them. A realization of the potential danger of this type of algal growth to humans is evident in the following recommendation made by the Texas Water and Research Foundation in 1955 [64]:

Although some of the organisms that produce taste and odor compounds in surface water supplies have been isolated and studied, the toxic qualities of the organisms have not been determined. If such organisms do contain toxic agencies, means or methods should be found which would destroy the organisms, either in the purification plant or in the distribution system.

As early as 1933 concern was expressed in Texas for the possibility of the presence of toxins in water. It was noted that the matter of toxins was receiving some attention and that activated carbon appeared to work very satisfactorily in their removal.[65]

Research on the toxic properties of the blue-green algae is now in progress at Baylor University, Waco, Tex. One part of the investigation has involved a study of an outbreak of algal poisoning of wild and domestic animals which were in contact with water in a stock pond near Waco. The outbreak occurred in August 1956 and reappeared during the same month in 1957 and 1958. The poisoning was attributed to the blue-green alga *Nostoc rivulare* which had not previously been reported as toxic.[66]

Detection of Interference Organisms

If undesirable organisms are to be adequately controlled, accurate records concerning their location, abundance, and effects are required. Frequent inspection of the raw-water supply, drainage area, treatment plant, and distribution system is essential. In addition, laboratory analyses of water samples to determine plankton types and numbers are important as well as tests to determine the type and intensity of tastes and odors in the water.

For the San Jacinto River Water System at Houston, a biologic survey

of the water was conducted over a period of several months. The data were sufficient to indicate that the water, when impounded, would support prolific biologic growths. This survey was made in order to arrange for remedies for undesirable conditions before acute problems developed.[67] The system of analysis followed at Dallas included weekly plankton examinations with arrangements for exchange of monthly reports with nine other water treatment laboratories.[68]

Kruse [69] worked out practical counting procedures for Tennessee River reservoir supplies in Alabama and nearby states. He summarized the purpose of plankton counting as to explain:

1. The cause of taste and odor and to indicate the methods of control

2. The cause of color

3. Clogging of pipes, screens, or filters

4. Unusually high chlorine demands

5. Cause of high pH which may result in poor coagulation

6. The loss in efficiency of condensers and other industrial equipment.

In Texas the Sedgwick-Rafter method for concentrating and counting samples has commonly been used.[18] Plankton counts have been recorded in areal standard units at Oklahoma City [20] and in northern Alabama.[69] At Laredo, Tex., the algae were recorded as the number per milliliter,[50] whereas Silvey used the number of filaments per liter in recording forms such as the diatom, *Melosira*.[19] In the USPHS water quality data program, plankton are listed as the number of "clumps" (isolated cells and colonies) per milliliter.[28]

The threshold odor test is being used at a number of treatment plants in the area. At Dallas, odor testing is reported to have been a part of the daily routine for many years, with excellent results in detecting any odors in the finished water. It has been considered the most important single test made in the laboratory for control purposes.[19]

Comparatively few accounts are available on the routine inspection of water supply reservoirs for biologic information on algal blooms, marginal growths, emergence of midge flies, fish kills, drainage or seepage of nutrients into the impounded waters, and areas where taste-and-odor organisms may be developing. In many areas the tendency has been to wait until a biologic problem has become acute before measures for control are initiated.[26] Cohen complained, in 1927, that the general instruction and preparatory training of the average plant operator in Texas did not include microscopical work, and usually the first knowledge that he had of a large increase in the plankton would be the evidence of increased color in the water or complaints from the consumers concerning tastes and odors.[51]

Preventive Measures

Attention has been given to the preparation of areas that are to be impounded in order that biologic growths and tastes and odors may be kept to a minimum. At Little Rock, detailed specifications for clearing the new reservoir site of trees, stumps, brush, vines, and any other vegetation more than 6 in. in height were formulated. Any material, such as manure, garbage, and sawdust piles, which would serve as sources of nutrients for growths in the water also was removed or buried.[70] In Alabama, regulations require that a basin be completely cleared prior to impounding. This practice was originally initiated

as a measure for the control of malaria.[71] The Texas and Arkansas departments of health have recommended specific measures for the sanitary control of watersheds.[72, 73]

Swamps present the greatest difficulties in the preparation of a catchment area. Seepage swamps contain ground water which has large amounts of carbon dioxide and nitrates, which are aids to the growth of aquatic vegetation. For rainwater swamps the construction of ditches is recommended to eliminate the excess water. Backwater swamps can be eliminated by regulating the main channel so that the water in the swamp is lowered. Seepage swamps are more difficult to control. Marginal drains can sometimes be used to intercept a large part of the seepage water.[46]

A number of practices are in use or have been recommended to control algae and other aquatic organisms in rainwater reservoirs. By intakes at various levels, the least objectionable water can be selected. When floating blue-green algae are abundant, as much as 75 per cent of them may be prevented from entering the treatment plant by taking water at some distance below the surface. At other times the upper water may contain fewer organisms than the lower layers.[25]

Results of an intensive biologic program for Lake Houston, Tex., made it evident that the plankton concentration was highest at the bottom, regardless of depth, during the day. This zone of maximum concentration would rise as dusk approached, scatter at night, and regroup at the bottom at dawn.[74]

For control of the larger aquatic plants as well as many of the smaller forms, the elimination of shallow areas of the reservoir is necessary. This may

be done by dredging, draining, or by control of dams to keep the water level up. Cutting or dragging is also used to destroy the larger plants.[46] Sanitary measures should be carried out with special attention to such matters as control of dead animals, garbage dumps, and sewage effluents in the reservoir drainage area.[42]

Establishment of biologically balanced conditions to reduce the problems caused by the overdevelopment of any one group or kind of aquatic life is advocated. For control of chironomids the reservoir should be stocked with fish, as chironomid larvae are a favorite fish food.[54] Recommendations for the establishing of biologically balanced conditions in the reservoir include prevention of pollution with additional nutrients, holding a predetermined water level, stocking with appropriate fish, and encouragement of fishing to reduce the release of organic matter when fish die off during winter and early spring.[40]

For large growths of algae in the reservoir, Fort Worth used a technique of control that appears to have been unique for Texas. Excess water for flushing purposes was drawn from Eagle Mountain Lake to wash the algae from Lake Worth into Trinity River.[40]

Chemical Controls

Copper sulfate has been in common use as an algicide for treating impoundments ranging from small ponds to large reservoirs. Less than 1.5 lb/mil gal has been indicated as sufficient to destroy *Asterionella*,[19] whereas other workers have recommended as much as 8 lb/mil gal for control of all algae without danger to livestock, humans, and fish.[75] Cohen stated that for Texas waters the range might be

1–33 lb/mil gal, depending upon the types of algae present.[28] Billings[31] claimed that, in general, copper sulfate was effective on the "higher" types of algae and diatoms, whereas chlorine was better on types which "border on bacteria."

Treatment with algicides has been considered effective when properly carried out.[76] For the six-state area the exact and proper procedure would apparently have to be determined separately for each situation. At Tyler, Tex., it was recommended that the entire lake be treated with 0.2 ppm of copper sulfate during early spring to slow down the growth. During the summer, copper sulfate was to be added around the margin of the lake once each month.[38] Shreveport, La., arranged for seasonal treatment of Cross Lake with copper sulfate as required according to the extent of algal growth. In May and June of one year, a total of 12,500 lb was applied to the lake but resulted in practically no reduction in the odor intensity due to algae.[35] On Lake Dallas, when copper sulfate was used to control *Melosira,* which was the major offending diatom, the organism was destroyed but a bloom of other algae greener than before developed.[40] As early as 1933, Cohen stated that prevention of maximum algal growth should be the rule, rather than its destruction later.[26]

A considerable amount of experimentation has been carried on in the south-central states with the use of chlorine and chlorine combinations to control algae and iron bacteria as well as tastes and odors. Experience at Laredo indicated that chlorination has to have time to act on diatoms if they are later to be caught in the floc particles during coagulation. Water entering the pre-liminary settling basin was treated with 26 lb/mil gal chlorine, which was also sufficient to carry an adequate chlorine residual through the plant and into the distribution system.[50]

Cohen stated that chlorine was valuable for control of algae where copper sulfate was not practical. His experiments at Lufkin, Tex., indicated that 0.05 ppm chlorine inhibited *Oscillatoria* and diatoms for a period of 8 days, and 0.5 ppm killed the *Oscillatoria* in 30 min. It took 1.5 ppm to kill *Spirogyra* in 3 hr. In general, a chlorine dose of 1.1 ppm was found to be successful for control of algae by treatment at the intake.[82] At Dallas, prechlorination was found to free the settling and mixing basin walls of a great deal of algal growth and to increase materially the length of filter runs.[77]

For the San Jacinto River water supply at Houston, prechlorination was reported to be used for color removal and destruction of substances producing taste and odor.[78] To eliminate tastes and odors at Tyler, it was found necessary to keep a 0.8–1.0-ppm chlorine residual in the water just prior to filtration. Superchlorination, representing a dosage of 7 ppm used immediately after addition of the coagulant, eliminated the taste and odor, whereas 3 ppm only aggravated the situation.[38] Superchlorination was ineffective in controlling tastes and odors at the Turtle Creek plant in Dallas, where the water was softened and had a high pH, whereas at the White Rock filter plant in the same city superchlorination reduced the taste to a mere trace.[39]

At Shreveport, La., chlorine-ammonia treatment of the raw water did not produce the odor-bearing compounds experienced with superchlorination. Chlorine dioxide produced no re-

ductions in odors.[35] New Orleans, La., has used chlorine-ammonia because of occasional occurrence of chlorophenol tastes with chlorine alone.[79] The director of sanitation at Beaumont, Tex., reported that the chlorine-ammonia process at a public outdoor swimming pool was satisfactory bacteriologically but that algae of a wide assortment flourished in spite of a 0.8–1.0-ppm chlorine residual.[80] Chlorine-ammonia was found to be better than chlorine alone for controlling algae in swimming pools at Fort Worth.[81]

Wichita Falls, Tex., reported that chlorine-ammonia produced as satisfactory results as did chlorine alone—and with a considerable saving of chlorine as well as complete freedom from chlorinous tastes. As an algicide, chlorine-ammonia had that property of sustained effect which chlorine alone did not have.[56] Little Rock, Ark., found that chlorine-ammonia treatment at the intake for the raw water improved the rate of flow in the raw-water line by reducing the layer of organic slime growth on the inside walls of the pipe. No tastes and odors were produced.[53]

Better results with chlorine as an algicide have been reported when a small amount of lime was added at the time of chlorination.[77] Chlorinated lime, dragged through the water in sacks, was recommended by Wisenbaker[46] for control of algae in reservoirs.

In an attempt to eliminate chironomids in the Church Point, La., reservoir, a dosage of 50 ppm chlorine was used with a contact time of 2 days. The larvae appeared again several months later, indicating the necessity for retreatment or additional methods.[84]

For impounded waters used primarily for fishing, duck hunting, and other recreational uses, weed killers have been used frequently. In Alabama, 2,4-D has been effective in controlling water hyacinth[61]; rosin amine D acetate and sodium arsenite have been tested for destruction of *Pithophora*[17, 60]; DDT has been used as a larvicide, but usually only on larger lakes, because of the expense.[71] At Lake Austin, *Myriophyllum* was prevented from growing by treatment with borax at the rate of 15 lb per 100 sq ft.[59] Experiments by the Texas Game and Fish Commission indicated that 2.5–8.0 ppm sodium arsenite was effective and cheap for the control of weeds, whereas chlorophenol dimethylurea (CMU) was successful in the control of marginal vegetation, and isoproply esters of 2,4-D were effective on cattails and other emergent vegetation.[58]

For small fishing lakes in Alabama and Mississippi, periodic applications of enough fertilizer to maintain a continuous bloom of algae at the surface are recommended. This bloom not only nourishes immense numbers of small animals upon which fish can feed, but it shades the bottom of the pond, thus preventing the growth of the larger attached aquatic plants.[82, 83]

Use of Activated Carbon

Activated carbon is in common use at many treatment plants throughout the area.[43, 44] Although its primary use is that of taste-and-odor removal, it has been considered significant in color removal, as an aid in settling, in sludge stabilization, and in toxin removal.[65] To eliminate tastes and odors, activated carbon has been introduced just ahead of the filters,[47] 20 min before coagulation, immediately after floc formation,[45] in the raw-water line at or near the intake,[84] or simultaneously and continuously at four separate points to guard against the sudden appearance of

strong odors.[28] The dosage has varied from as low as 2 ppm [84] to 30 ppm,[85] the removal of tastes and odors ranging from slight to complete. In some cases the improvement was temporary unless the treatment was repeated at a second location as the water passed through the treatment plant.[45] At some plants the carbon has been applied only when necessary, as, for example, at Little Rock,[85] and at other plants it has been applied continuously, as at the North Texas Municipal Water District Plant.[84]

Other means used in this area to remove malodorous substances are aeration,[86] chlorination, chlorine dioxide treatment, and potassium permanganate treatment.[18] Aeration was reported as sometimes killing microorganisms and causing odors through the liberation of oils from the dead organisms.[87]

Mechanical Controls

Mechanical practices for reducing algal interference in water treatment or for reducing the tastes and odors in the treated water are numerous. These range from "bumping" a filter to covering of a reservoir.[88] Winston recommended that the dirt layer be scraped from the surface of a filter as a practical method in small treatment plants.[89] Correct placement of the foot valve in the sedimentation basin, painting the walls of the basin with whitewash from the surface to 3–4 ft below the water level, and frequent and regular removal of the sludge from the bottom of the basin are considered important in the control of algae and tastes and odors.[90]

Improvements and frequent adjustments in the method of coagulation are advocated, as that process, as practiced in many plants, frequently reduces the organic content of the water by only about 50 per cent.[91]

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Observations and Studies of *Crenothrix polyspora*

———————Ralph S. Wolfe———————

A contribution to the Journal by Ralph S. Wolfe, Assoc. Prof., Dept. of Microbiology, Univ. of Illinois, Urbana, Ill. The study was supported by Grant 6430 from USPHS.

I N 1870, Ferdinand Cohn described a filamentous bacteria-like organism that he named *Crenothrix polyspora*.[1] His description of this organism included a series of line drawings reproduced here as Fig. 1. To date, these drawings are still the most careful study of *Crenothrix* ever done.

Present Observations

In a recent article in the JOURNAL,[2] which described some of the author's work with iron bacteria, a request was made for samples of iron sludges suspected to contain *Crenothrix polyspora*. In response to this request, Arne Zimmergren, an engineer of Malmo, Sweden, kindly provided some ferric hydroxide sludge from the filters of the water plant in Malmo. The sludge contained several colonies of *Crenothrix polyspora*. He also supplied several photomicrographs that supported his observation that colonies of *Crenothrix* may not be encrusted with ferric hydroxide, even though they may be found in a sludge of ferric hydroxide. The organism from Sweden looked very much like the one illustrated by Cohn.

At the Richmond Water Works Corp., Richmond, Ind., excellent colorless colonies of *Crenothrix polyspora* have been found associated with clumps of brown, hairlike bacteria from filter galleries. Such a colony mounted in water is illustrated in Fig. 2a. Reflected light was used to accentuate the colorless nature of the colony at low magnification. On each side of the flamboyant colony are filaments of the brown hairlike organism which are poorly defined because they do not reflect the light. In Fig. 2, b–h represent photomicrographs obtained by dark-phase contrast microscopy of living specimens mounted in water.

Samples of *Crenothrix polyspora* from Sweden and Indiana looked alike. In neither instance were the filaments visibly encrusted with iron or manganese, and they appeared analogous to cellophane tubes. When the colonies were treated with dilute hydrochloric acid and ferrocyanide solutions, a few filaments in each colony exhibited a weak Prussian blue reaction near the base of the filament, indicating a slight absorption of ferric ions.

Comparative Study

The only comparative study of *Crenothrix* made in America seems to be the one by Kolk.[3] This study was made in 1937, in an area of Brooklyn, N.Y., that received hard water supplied from artesian wells. Kolk found that the sheath of *Crenothrix* was colorless throughout its entire length, and, although not thick, was distinctly visible at both the apex and base of the filament. Most of the photomicrographs
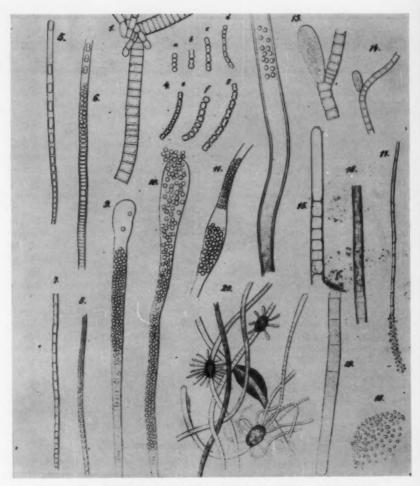
Fig. 1.   Reproduction of Cohn's Original Illustration of *Crenothrix polyspora*

Some of the drawings shown are recommended for comparison with similar forms shown in Fig. 2: Compare 4 with Fig. 2e and 2f; 5 and 6 with Fig. 2c and 2h; 7 and 8 with Fig. 2g; and 9 and 10 with Fig. 2d.   The forms illustrated in 13–19 have not been reported by later observers.   (Reproduced with the permission of Duncker & Humblot, Berlin-Lichterfeld, Germany.)
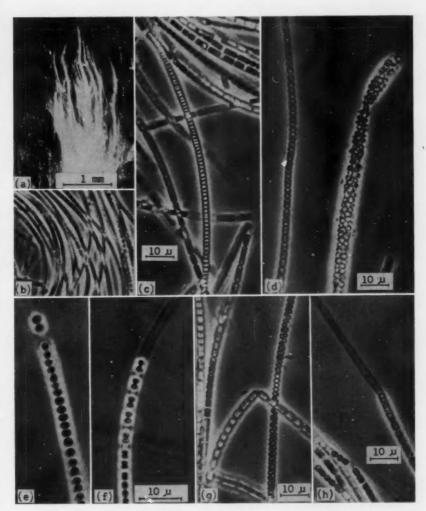
Fig. 2. Photomicrographs of *Crenothrix polyspora*

Figure 2a shows a colony of Crenothrix *mounted in water and photographed by re-flected light.   The absence of encrusted filaments may be noted.   Figures 2b–h show phase contrast photomicrographs of living specimens mounted in water.*

published by Douchon and Miller in their study of the effect of chemical agents on iron bacteria [4] do not show iron encrustation of the sheaths of *Crenothrix*. Harder also mentions that the sheaths of *Crenothrix* are generally colorless.[5]

On the basis of past as well as present observations, it appears that healthy, growing colonies of *Crenothrix polyspora* may not actively deposit iron or manganese in the sheaths. To obtain more evidence and to study *Crenothrix* in greater detail, the author would be pleased to examine additional samples of water and iron or manganese sludges that may be suspected of containing *Crenothrix polyspora*. The original drawings of Cohn and the author's photomicrographs are presented here so that the nature of the organism may be clear.

The distinguishing characteristic of *Crenothrix polyspora* is the production of spherical, nonmotile cells called conidia. These may be formed in a single filament of normal size, as shown best in Fig. 2e and 2f, or they may be produced in large numbers in an expanded sheath, as shown in Fig. 2d and 2g. In Fig. 2f and 2h, the empty sheath of a filament is apparent. The great variation in cell shape is evident throughout the photographs. Unless the spherical cells (conidia) of *Crenothrix polyspora* are actually observed, it is very easy to confuse this organism with *Sphaerotilus natans,* which never forms conidia. To the author's knowledge, *Crenothrix polyspora* is the only presently recognized species of *Crenothrix,*

and it has never been obtained in pure culture. Studies of pure cultures, however, have shown that large amounts of iron and manganese may be deposited in the sheaths of species of *Sphaerotilus* [6]; it remains to be demonstrated with pure cultures that the same is true for *Crenothrix*. There is little, if any, precise evidence to support the popular belief that *Crenothrix* is an iron bacterium.

Acknowledgment

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Suggested Method for Iodide Determination

—————John R. Rossum and Primo A. Villarruz—————

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THE iodide concentration of ground water is one of the best criteria for the indication of contamination from connate brines. Many fresh-water aquifers overlie these brines, which are often associated with oil- or gas-bearing formations. Contamination of the fresh water may occur from defective oil or gas wells. Early indications of the presence of connate waters could make corrective action possible before extensive deterioration of the quality of fresh water has occurred.

Chemically, connate brines may be described as highly mineralized calcium chloride or sodium chloride waters. The total salt concentration of these waters may exceed that of ocean water by a factor of two or more. Connate waters often contain appreciable concentrations of methane, sulfide, and boron; occasionally, such unlikely constituents as manganese and arsenic are found in abnormally large concentrations. Invariably, the iodide-to-chloride ratio in connate waters is greater than that in ocean water by a factor of approximately 100.

The potential value of iodide determinations has not been fully realized because of the lack of a convenient method of analysis. The method in the tenth edition of *Standard Methods*[1] requires special equipment and reagents not found in most water laboratories and involves a time-consuming distillation procedure to eliminate interfering ions. The procedure given in earlier editions is exceedingly tedious and grossly inaccurate. The result has been that few laboratories made iodide determinations, and routine iodide tests on ground water samples were impractical.

The following modification of the method of Lein and Schwartz[2] takes less than an hour, and, if several samples are tested in batches, the average time per sample is only a few minutes.

1. General Discussion

1.1. *Principle.* Iodide catalytically accelerates the reduction of ceric ion by arsenious ion. Solutions of ceric ion are yellow; solutions of cerous ion, colorless. The rate of fading of the yellow color is a function of the iodide concentration and may be conveniently determined with a photometer. An increase in temperature not only materially speeds the reaction but also intensifies the color of the ceric sulfate solution. Unfortunately, these effects do not tend to cancel each other. With small iodide concentrations, the increase in ceric ion color more than offsets the increased rate of fading; with greater iodide concentrations, the increased rate of reaction completely overshadows the increase in color. It is essential that the temperature of the standards and the unknowns vary from

one another by not more than a few tenths of a degree.

The rate of reaction decreases with increasing hydrogen ion concentration. In order to minimize the effect of alkalinity of the water samples on the final pH of the reaction mixture, the reagents should contain an excess of sulfuric acid. The ceric sulfate solution should contain the same concentration of sulfuric acid as the solution to which it is added. Otherwise, the heat liberated by dilution of the sulfuric acid would increase the temperature, and erratic values could result from local variations of temperature within the solution caused by slight differences in the way the solutions are mixed. The sulfuric acid is diluted so that most of the heat of dilution is dissipated prior to the mixing of the solutions with the water sample.

When the iodide concentration is small, it is necessary to measure a small difference between the fairly intense color of the reagent blank and the color of the sample. The percentage of error can be very great if initial ceric ion concentration present in the samples and standards varies appreciably. It is essential that all reagents and samples be added to the reaction vessels in as nearly the same volume as possible.

Care should be taken to avoid contamination of the samples and the glassware. Iodide is adsorbed rather tenaciously to glass and is not readily removed by rinsing with water. Because it stays on the walls of the reaction vessels, iodide can be a serious source of error. It is therefore important that vessels be thoroughly cleaned before each determination by a rinsing with concentrated hydrochloric acid, followed by two rinsings with distilled water.

1.2. *Interferences.* Reducing agents that react rapidly with ceric sulfate and cause false positive tests are eliminated by adding dilute permanganate. If reducing agents were not eliminated, the magnitude of the resulting error would be independent of iodide concentration, so that the percentage error would be very large for small iodide concentrations. Oxidizing agents cause negative results by lowering the concentration of arsenious acid. The magnitude of the error is proportional to the iodide concentration. A concentration of 10 mg/l chlorine, or its equivalent, was found to cause less than a 1 per cent error.

Mercury causes a negative error, which increases with increasing iodide

TABLE 1
Concentrations of Ions Causing No Interference

Ion	Concentration mg/l
Chloride	5,000
Sodium	3,000
Alkalinity (as $CaCO_3$)	1,000
Nitrate	750
Calcium	250
Cadmium	10
Bromide	5
Cyanide	5
Fluoride	5
Silver	1
Iron	1

concentration, and cannot be tolerated in concentrations exceeding 0.1 mg/l. Chaney [3] states that osmium interference is ten times as serious as mercury interference.

A number of ions cause no interference when tested in the concentrations shown in Table 1. Recovery tests (see Sec. 6) indicate that ions normally present in natural waters do not interfere.

2. Apparatus

2.1. *Colorimetric equipment.* One of the following is required:

a. *Spectrophotometer,* for use at 420 mμ, providing a light path of approximately 1 cm.

b. *Filter photometer*, equipped with
a blue filter having a maximum trans-
mittance of nearly 420 mμ and provid-
ing a light path of approximately 1 cm.

2.2. *Timer*.

3. Reagents

3.1. *Acid-permanganate   solution*.
Cautiously add 200 ml concentrated
H_2SO_4 to approximately 600 ml water.
Cool to room temperature. Dissolve
0.025 g potassium permanganate
($KMnO_4$) in this solution. Make up
to 1 liter. The permanganate is gradu-
ally reduced. A fresh solution should
be prepared each week.

3.2. *Salt-arsenite solution.* Dissolve
80 g NaCl and 1.30 g $NaAsO_2$ in
water, and make up to 1 liter. This
solution is stable indefinitely.

3.3. *Ceric sulfate solution*. Cau-
tiously add 57 ml concentrated H_2SO_4
to about 600 ml water. Dissolve 4.24 g
$Ce(HSO_4)_4$ in this solution. Cool to
room temperature, and make up to 1
liter. This solution is stable indefinitely.

3.4. *Stock iodide solution*. Dissolve
1.31 g dry KI in water, and make up
to 1 liter. This solution contains 1,000
mg/l iodide and is stable indefinitely.

3.5. *Working iodide solution*. Di-
lute 5 ml of the stock iodide solution
to 500 ml. Dilute 5 ml of the resulting
solution to 500 ml. The second solu-
tion contains 0.100 mg/l iodide. There
is no appreciable loss of iodide from
this solution after 1 month.

3.6. *Hypochlorite solution* (required
only for the alternative procedure given
in Sec. 4.6). Prepare a solution con-
taining approximately 0.1 per cent
available chlorine from commercial
bleach.

4. Procedure

4.1. Prepare standards containing
0.000, 0.020, 0.040, 0.060, 0.080, and
0.100 mg/l iodide from the stock solu-

tion. Fewer standards may be used,
except on the first determination (see
Sec. 5). Pipet 5 ml of each standard
and 5 ml of each sample into clean,
dry 30-ml test tubes. Samples having
iodide concentrations greater than 0.1
mg/l are diluted to fall within the range
of the standards.

4.2. Pipet 5 ml of acid-permanganate
solution into each test tube. If the
purple color disappears, an excessive
quantity of reducing agent is present,
and the analyst must resort to the dis-
tillation procedure outlined in *Standard*

TABLE 2

Results of Recovery Tests

Test	Iodide Found—*mg/l*		
	Tap Water	Tap Water Plus 0.0375 mg/l Iodide	Tap Water Plus 0.075 mg/l Iodide
1	0.0131	0.0504	0.0915
2	0.0141	0.0504	0.0915
3	0.0136	0.0504	0.0915
4	0.0141	0.0504	0.0893
5	0.0141	0.0514	0.0882
6	0.0151	0.0524	0.0893
7	0.0146	0.0504	0.0893
8	0.0141	0.0524	0.0893
9	0.0146	0.0514	0.0904
10	0.0146	0.0504	0.0904
Average	0.0142	0.0510	0.0901
Iodide recovered— mg/l		0.0368	0.0759
Standard deviation	0.00062	0.00085	0.00119
Recovery—%		98.14	101.16
Coefficient of varia- tion—%	4.38	1.66	1.31

Methods,[1] unless the reductant can be
removed or oxidized without loss of
iodide.

4.3. Add 5 ml of salt-arsenite solu-
tion to each test tube. Mix, and allow
to stand in a test-tube rack until all
solutions are at the same temperature.

4.4. At 1-min intervals, pipet 5 ml
ceric sulfate solution into each test tube,
and mix immediately after the addition
of the reagent.

4.5. Wait 10±0.2 min after the ad-
dition of the ceric sulfate reagent, and
read the color of ceric ion remaining

in each test tube. The photometer is adjusted to zero absorbance with water at a wavelength of approximately 420 mμ. The reaction time must be the same for standards and samples, although it may be increased to 15± 0.3 min with a corresponding increase in sensitivity. As the tubes must be at the same temperature during the reaction period, exposure to the heat of the photometer lamp should be minimized. If photometer readings cannot be conveniently made at 1-min intervals, the alternative procedure outlined in Sec. 4.6 may be used.

4.6. Wait 10±0.2 min after the addition of the ceric sulfate, and pipet 5 ml hypochlorite solution into each tube. This stops the reaction by oxidizing the arsenite. Photometer readings are then made at the analyst's convenience, for time is no longer a critical factor. The temperature should not vary, however, because the color of the ceric ion is dependent on temperature.

5. Calculations

Prepare a curve by plotting the iodide concentrations of the standards against the corresponding absorbances, and read the iodide concentrations of the samples from this curve. Beer's law is not obeyed, and small changes in temperature will materially shift the curve. Once the curve has been drawn, however, it may be used in conjunction with the following equation to calculate the iodide concentration of unknowns (two standards containing, respectively, 0.000 mg/l and 0.100 mg/l iodide are run along with the sample) :

$$R = a - \frac{(b-c)\ (a-d)}{(b-e)}$$

in which R is the reading that the sample would have had, had it been tested together with the standards used for plotting the curve, so that the iodide concentration may be read directly by use of this value ; a, the absorbance of 0.000 mg/l iodide from the curve ; b, the absorbance of the 0.000 mg/l standard ; c, the absorbance of the sample ; d, the absorbance of 0.100 mg/l iodide from the curve ; and e, the absorbance of the 0.100 mg/l standard.

6. Precision and Accuracy

Ten replicate tests were made on each of three samples : tap water and the same sample of tap water with 0.0375 mg/l and 0.075 mg/l iodide added (Table 2). The figures in the table indicate that the method is capable of a precision of approximately 3 per cent, or 0.0015 mg/l with a confidence of 95 per cent. Mechanical errors in plotting the calibration curve may be expected to reduce the accuracy of the results by an additional 2 per cent. When all factors are taken into account, results within 0.002 mg/l in the range of 0–0.040 mg/l, or 5 per cent in the range of 0.040–0.100 mg/l, may be expected 95 per cent of the time. The modification with hypochlorite is less precise. Errors are approximately 1.5 times those for the regular procedure.

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Electrode for Simplified Field Determination of Chloride in Ground Water

────────────William Back────────────

A contribution to the Journal by William Back, Geologist, USGS, Washington, D.C. Publication of this article is authorized by the Director, USGS.

IN many ground water investigations it is desirable to have a convenient field measure of the chloride concentration in the water as an indication of its chemical character. The data gained from field determinations are useful in a number of ways, as, for example, in mapping the extent of salt water encroachment or as a guide for more selective sampling of the aquifers within an area.

A newly developed silver–silver chloride electrode and a standard pH meter (shown in Fig. 1) provide a simple and accurate means for the field determination of chloride concentration. The electrode contains a billet of silver chloride, with a large part of its surface area exposed to the sample. In this respect it differs from earlier electrodes, which exposed only a small filament of silver chloride and were not rugged enough for field use. The new electrode is used with a saturated calomel reference electrode.

Procedure

Any sensitive pH meter on which millivolts can be read can be used to measure the electrical potential, or electromotive force, in any solution in which the silver–silver chloride electrode is immersed. The meter is calibrated by measuring the potential in a series of standard solutions of known chloride concentration. The millivolt readings are plotted on the arithmetic scale and the corresponding known chloride concentrations on the logarithmic scale of semilogarithmic paper. This plot provides a standard curve on which the potential, in millivolts, as read for an unknown solution, can be used to determine the chloride concentration.

Instructions from the manufacturer suggest calibrating or standardizing the meter and electrode at least once a day. In the field, the electrodes were standardized before and after the analysis of each group of unknown samples. The standardizations of both Jul. 27 and Jul. 30 were sufficiently close to permit the use of the same standardization curve. From the standardization two facts were established: (1) the cumulative drift in accuracy from the amount of chloride present in 25 samples was within 1 per cent and (2) in 3 days of storage, the drift in the instrument was within 1 per cent. Jones and Kehoe[1] found the instrument sufficiently stable that weekly standardizations gave an accuracy of ±10 per cent of the amount of chloride present.

To obtain the set of measurements described below, ground water samples were collected and allowed to stand until they were at the same temperature as the standard solutions. A

standard curve was then prepared for the meter and electrode, and the potential in the ground water samples was measured. The electrodes were dipped into a beaker of distilled water between measurements of unknown samples. The measurements were made very quickly, at the rate of about 20 samples per hour, after standardization. The electrodes were left in each solution, however, until they had apparently stabilized.

ments made at solution temperatures of approximately 90°F.

Results of Measurements

Fifty samples of water, ranging in chloride concentration from less than 10 ppm to more than 20,000 ppm, were collected. After the chloride concentration was measured in the field, the 50 samples were analyzed in the laboratory by the Mohr titration method for dilute samples and by the gravi-
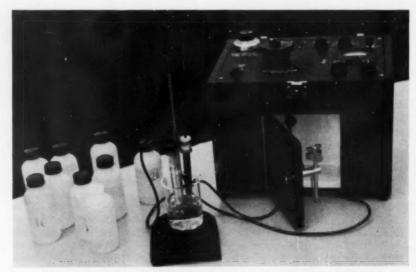
Fig. 1.   Equipment for Field Determinations of Chloride in Ground Water

Shown are the pH meter and silver–silver chloride electrode as set up for field determinations.

Because the solubility of silver chloride is affected by temperature, it is desirable to have the unknown solutions at the same temperature as the standard solutions before measuring the potential. Figure 2 shows the effects of temperature on the position of the standard curve for solution temperatures of 75 and 95°F. The standard curve shown in Fig. 3 was drawn from measure-

metric method for samples containing more than 200 ppm chloride. The values obtained under conditions of routine analysis are accurate to about ±4 per cent of the true value. Curve *A* in Fig. 3 was obtained by measuring the known standard solutions in the field. To develop Curve *B*, the laboratory values of chloride concentration for the 50 water samples were plotted

against the corresponding electrical potentials measured in the field. Field determinations for 70 per cent of the samples are within 20 per cent of the value found in the laboratory. The field values average slightly less than 9 per cent higher than the laboratory values.

Theory of Measurements

The theory supporting field measurements of this type is discussed here only briefly because it is well documented in such standard references as Kolthoff and Furman [2] or Kolthoff and Laitinen.[3] The sensitive part of the electrode is made from a mixture of metallic silver and solid silver chloride. When immersed in water the reactions at the electrode are

$$Ag^0 = Ag^+ + e^-$$

and

$$AgCl = Ag^+ + Cl^-$$

By subtraction, the overall reaction is

$$Ag^0 + Cl^- = AgCl + e^-$$

By use of the Nernst equation, the potential developed by the electrode and the ions it can put into solution is:

$$E = E^0 - \frac{RT}{nF} \log \frac{1}{a_{Cl^-}} \dots\dots\dots(1)$$

in which E is the measured potential, E^0 is the standard potential of the reaction, R is the gas constant, T is the absolute temperature of the solution, n is the number of electrons, F (faraday) is 96,500 coulombs per gram equivalent, a_{Cl^-} is the effective or thermodynamic concentration of Cl^-. Upon substitution of the appropriate data, Eq 1 becomes

$$E = E^0 + 0.06 \log a_{Cl^-} \dots (2)$$

The relationship in Eq 2 shows that E, the potential measured in the field, is controlled by E^0, the standard poten-

tial of the chemical reaction, and a_{Cl^-}, the concentration of the chloride ion.

Errors and Limitations

The manufacturer's instructions state that the electrode should not be used in solutions containing ions that form less soluble silver compounds than silver chloride, as, for example, sulfide or bromide. Perhaps some of the error is due to the presence of these or other ions. Another source of error, which may not have been fully appreciated during these field studies, is inaccuracy of potential readings obtained, owing to the delay in response of the elec-
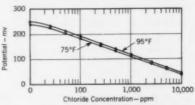
Fig. 2. Effect of Solution Temperature on the Relationship Between Electrical Potential and Chloride Concentration

As can be seen, the solution temperature has a definite effect on the position of the standard curve.

trode when measuring successive samples differing greatly in chloride concentration. Also, part of the error is due to the gradual contamination of the surface of the electrode during the analysis of a set of samples. A new surface of the electrode is exposed by gently grinding the tip of the electrode with fine emery paper. It was found to be good practice to rejuvenate the electrode in this manner before each standardization to avoid erratic readings. The electrodes cannot be used in metal-cased wells, as close proximity of metal gives erroneous readings.
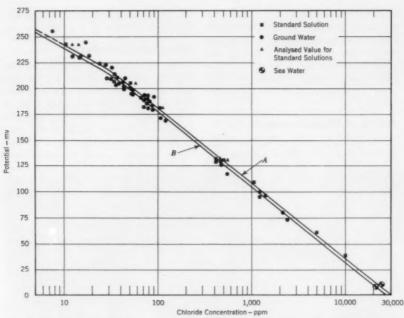
Fig. 3. Comparison of Field and Laboratory Determinations of Chloride Concentration

Curve A is a standard curve plotted from field measurements of standard solutions at 90°F. Curve B is a computed-mean curve of laboratory values for chloride concentrations.

Conclusion

The advantages of this method over the previous methods are its simplicity and the short time required for running the tests. The results are well within the range of accuracy required for field determinations. The equipment is light in weight and readily portable. The only equipment needed in addition to the pH meter is the silver–silver chloride electrode, which costs $45.

Acknowledgments

Thanks are due the following members of USGS: R. H. Brown, research engineer, and Ivan Barnes, geologist, for their guidance and helpful criticism; G. G. Magin, chemist, for his assistance in the laboratory; and G. C. Wyrick, geologist, for his assistance in the field.

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Studies on the Toxicity and Decomposition of Fluosilicic Acid

Robert S. Ingols

A contribution to the Journal by Robert S. Ingols, Prof. in Applied Biology, Georgia Institute of Technology, Atlanta, Ga.

BECAUSE of the increasing importance of various compounds of fluorine in water treatment, in the production of nuclear reactors, in the refinement of phosphate ores for fertilizer, and in the production of aluminum compounds from cryolite, it is believed that more should be known about the chemistry of fluosilicic acid. The author, for example, was asked to evaluate the toxicity of fluosilicic acid which was found in an industrial waste from a pharmaceutical plant processing cryolite.

Toxicity Study

A study of the toxicity of fluosilicic acid and fluosilicates [1] was initiated to determine the concentration of fluosilicic acid and its salts required to kill fish. A search of the literature revealed little chemical information except for a few reaction constants given in tables, which are not of much practical value. Sodium fluosilicate from different commercial sources did not appear homogeneous and was found to be quite insoluble. The solutions obtained after several hours had pH 4.0, which was unsatisfactory for the toxicity studies, because at that concentration, the hydrogen ion is itself too toxic to permit a study of the toxicity of other ions. Thus, it became necessary to work with the acid and some alkali to produce solutions with satisfactory pH values and a high enough salt concentration to be toxic, if possible.

As fluosilicates are used to fluoridate potable water supplies, besides being found in industrial wastes, it was felt a description of their chemistry might be useful to the sanitary engineering profession.

Chemical Studies

When a low $(0.0006M)$ concentration of the commercial fluosilicic acid was titrated potentiometrically, a smooth, monovalent-acid curve was obtained, as shown in Fig. 1, with a single inflexion point at pH 7.0. By using the equation

$$H_2SiF_6 + 2NaOH \rightarrow Na_2SiF_6 + 2H_2O \quad ..(1)$$

and an equivalent weight one half of the molecular weight, the data indicate an acid strength three times that claimed by the vendor. A subsequent plot of a second potentiometric titration with the acid concentration twenty times that used previously reveals (as shown in Fig. 2) a small but definite inflection at pH 3.4 (3.2–3.6), where one third of the total alkali required to obtain pH 7.0 had been used. It is now known that the first inflection point in Fig. 2 corresponds to the equivalence point of Eq 1. This would account for the pH of the commercial salts.
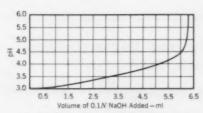
Fig. 1. Potentiometric Titration of 0.0006M Fluosilicic Acid

As can be seen, titration with the small volume of acid used produced a smooth, monovalent-acid curve with a single inflection point at pH 7.

When it became obvious that the reaction in Eq 1 did not correspond to the titration of the acid to pH 7.0, a need for more study was apparent. It was soon revealed that titration to pH 7.0 yielded a reaction in which six molecules of sodium hydroxide reacted with one molecule of fluosilicic acid to yield an equation which might be written:

$$H_2SiF_6 + 6NaOH \rightarrow$$
$$6NaF + Si(OH)_4 + 2H_2O\ldots(2)$$

It was found subsequently that this is the reaction used by the vendor for controlling the acid strength.

Determinations of the silica concentration in the fluosilicic acid solution by the heteropoly blue method [2] and of the fluorine concentration by the thorium nitrate titration method [3] indicated that the molecular relationships in Eq 2 must be correct. No precipitate of silicic acid was observed in any titrations of fluosilicic acid.

In the toxicity studies made by the author, Milan (Italy) tap water, rather than distilled water, was used for the fish medium. The significant salt in the Milan tap water is calcium bicarbonate in 0.002M concentration.

When fluosilicic acid was added to the tap water to produce pH 5.0, two important observations were made: (1) the fluosilicic acid in 0.5M solution (commercial 7.0 per cent solution) behaved as a 3N solution, and (2) no calcium fluoride precipitate formed immediately. (The solutions were observed for the entire 72-hr test period.) A calcium fluoride precipitate formed between 36 and 40 hr in one sample. Thus, the results indicate that fluosilicic acid behaves like a hexavalent acid while it is actually decomposing and producing fluoride ions very slowly at pH 5.0–7.0. A study of the solubility product for calcium fluoride and observations of the addition of an equivalent amount of sodium fluoride indicate that a precipitate of calcium fluoride should form, but apparently the hydrolysis of the fluosilicic acid is slow enough to prevent the formation of localized areas of excessive supersaturation.

It was decided to mix various amounts of sodium fluoride with fluosilicic acid and observe the pH and time for attaining pH equilibrium in
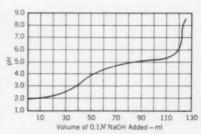
Fig. 2. Potentiometric Titration of 0.010M Fluosilicic Acid

The shift in the curve at pH 3.4 (one-third of the total alkali) is the endpoint of the formula acid (equivalence point of Eq 1).

the presence of various amounts of alkali. The data thus obtained indicated that fluosilicic acid and its salt were much slower in decomposing in the presence of sodium fluoride at pH values near 7.0. The effect of fluoride ion on the stability of the fluosilicate ion can be explained by a rapid exchange in fluorine between fluoride ion and the fluoride of the fluosilicate ion, as demonstrated by Atem and Bigot.[4]

Conclusion

It has been shown that fluosilicic acid and silicon tetrafluoride decompose rapidly above pH 3.6, although the rate of decomposition is retarded by the presence of extra fluoride ions. Either the rate of decomposition or the manner of decomposition can favor a supersaturated solution of calcium fluoride at pH 6.5 for a period of several hours at a $0.002M$ concentration. At any given pH, the toxicities of fluoride and fluosilicate ion are equivalent.

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Sealing of Joints in Concrete-Paved Reservoirs

—Henry Ward Tyler—

A paper presented on May 17, 1960, at the Annual Conference, Bal Harbour, Fla., by Henry Ward Tyler, Prin. Engr., Design & Plans Div., Eng. Dept., Seattle, Wash.

THE use of portland cement concrete for lining ground-level reservoirs results in maximum, useful reservoir life with minimum maintenance. Adequate reinforcement of the slabs to control cracking due to changing conditions of temperature and moisture and proper sealing of the construction and contraction joints to prevent leakage at the joints are the keys to a watertight reservoir.

Seattle's eleven concrete-paved reservoirs, constructed during the last 60 years, vary in capacity from 5 to 68 mil gal. From experience gained in the design and construction of these reservoirs, and through the use of new materials now on the market, an effective method of sealing joints has been adopted by the Seattle engineering department.

Progress in Joint Sealing

Methods of joint sealing have gone through a slow evolution. Twenty years ago, a strip of sheet copper was used for a waterstop. This was formed in a U shape at the joint to allow for expansion or contraction. It extended approximately 6 in. on either side and was bent up 1 in. at the ends. In Seattle, the copper strip was set at the bottom of the joint and was supported by a 4×12-in. concrete sill that was poured in place. The joint was filled with a strip of expansion joint material

and topped off with hot-poured tar or asphalt. Concrete slabs were unreinforced, and joints were spaced 12–15 ft apart in both directions. This resulted in excessive costs of forms and joint materials. Slabs cracked between joints in spite of this spacing and required patching every year.

It was thought that the copper seal was doing a satisfactory sealing job, until one reservoir was constructed without the hot-poured asphalt sealer. This reservoir leaked badly when it was first filled, and the water was drained out. A hot-poured rubber-asphalt joint sealer was then applied.

New products have changed the methods of joint sealing. Together with new techniques of concrete pouring and finishing, these methods have resulted in lower costs and better reservoirs. The present practice in Seattle is to use steel reinforcement in the slabs, which are poured 20 ft wide and as much as 66 ft long. This has resulted in fewer joints, controlled cracking in the slabs, and, with the new joint materials which will be described, less leakage.

Joints in the last reservoir completed in Seattle were constructed as contraction joints, with no provision for expansion. The theory was accepted that the concrete shrinks in setting and that subsequent swelling due to immersion in water, or expansion due to heat

within the ranges of temperature to which the concrete will be subjected in the reservoir, will not exceed the original volume. It is vitally important, therefore, to insure that the surface of the joint is properly sealed, so that sand or silt will not deposit in the joint when it contracts.

Effective Joints

There are two essential elements in an effective joint; a waterstop cast in the slab and a joint sealer at the surface (Fig. 1). The waterstop prevents the semiplastic sealer from being forced through the contracted joint under water pressure, and the sealer protects against leakage and keeps sand or silt from compacting in the contracted joint. Proper construction of these two elements will insure a watertight joint.

A number of good waterstops are on the market today. These usually consist of an extruded form of polyvinyl chloride or a form of molded rubber in continuous strips 4–9 in. wide. The waterstops either have bulbs at the ends (in dumbbell types) or multiple small ribs that wedge against the concrete when the material is stretched by joint contraction. Some of the waterstops are flat; some have a hollow bulb or a U shape at the center. Others are shaped to combine a waterstop with a keyway in the concrete. Waterstops with a hollow center bulb are generally designed for use where the greatest contraction is expected; flat strips without center bulbs will stretch within the material itself and are used where little or no contraction is indicated.

At joint intersections the polyvinyl chloride material is easily cut and spliced in the field, for a hot knife or a special electrically heated tool will melt the material, which permits weld-

ing of field splices. Waterstops made of molded rubber are provided with special rubber tees or crosses and rubber unions that are cemented or vulcanized.

Construction of Joint

In casting the waterstop in the slab, it is usually necessary to use a split form. Some waterstops, however, are made so that they can be tacked to a solid form. In forming the joint, a groove is left at the surface for the sealer. In Seattle, a wood strip, 1 in. deep and tapered from $\frac{5}{8}$ in. at the top to $\frac{1}{2}$ in. at the bottom, is used for this purpose. The wood strip is left in

Fig. 1.　Contraction Joint Seal

A represents the polysulfide rubber compound; B, concrete lining; C, reinforcement; and D, polyvinyl chloride waterstop. The seal for a construction joint is the same, except that a smaller waterstop without center bulb can be used.

place until all concrete construction has been completed and cleaned up. This keeps sand from lodging in the groove and also keeps curing compounds away from surfaces to which the sealer must adhere. After the concrete slabs are cured, some shrinkage will take place, and the wood strips will be easily removed. Also, the groove will have increased in width from $\frac{1}{8}$ in. to $\frac{1}{4}$ in. After the wood strips are taken out, the grooves are blown out with compressed air to remove all dirt and loose material before the sealer is poured.

Sealers

Two types of sealer have been used successfully: a hot-poured rubber-

asphalt mixture and a cold-applied polysulfide rubber compound. The rubber-asphalt is an efficient sealer when properly applied. Its advantage lies wholly in its lower initial cost. Its disadvantages are that it is difficult to pour on inclined joints, and it will flow under water pressure after it has set, so that it is essential to have it backed up with a good waterstop.

The polysulfide rubber compound comes in two parts, which are mixed on the job and used within a few hours. It has good working characteristics and, when cured, becomes a firm, rubber-like material strongly adhering to concrete and elongating to twice its original joint width without loss of sealing properties. By varying the two parts, the sealer can be made pourable and self-leveling for horizontal joints or heavy and putty-like for sloping joints. The pourable material can be applied from a container with a pouring spout; the heavy material, with a pressure gun or a putty knife. The surfaces of the joint must be clean, dry, and primed with a special primer a few hours before the sealer is applied. The primer is a thin liquid that can be applied by brush.

The polysulfide rubber compound costs, per gallon, several times more than the rubber-asphalt compound; but when the cost of labor is added, the net cost per linear foot of joint will not be substantially different between the two compounds. The results attained with the polysulfide rubber compound are worth the greater cost. Unlike the rubber-asphalt material, the polysulfide rubber compound will not flow under water pressure and has been used, in some instances, without the waterstop to back it up. This practice, however, requires caution. The waterstop can be installed only when the slabs are poured, but the joint sealer can be renewed if occasion demands. For the little extra cost of installing both, a tight and permanent job of joint sealing is insured.

Simulation of Filtration on Electronic Digital Computer

Kenneth J. Ives

*A contribution to the Journal by Kenneth J. Ives, Lecturer in Charge,
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THE purpose of filtering a fluid with a porous granular medium is to remove suspended material from the fluid. This is particularly true of rapid filters used in water treatment. Filtration is used also in industrial chemical processes, in removing material from air and gas, and occurs naturally when water flows into geologic strata.

The account given here is based on a study of water filters under conditions of one-dimensional mass flow (varying only with time and position) and a concentration of suspended material too small to affect the physical properties—viscosity and density, for example—of the fluid. For simplicity, the filter medium is assumed to be isotropic and homogeneous throughout its depth, and the suspended material is assumed to consist of discrete, unisize, homogeneously dispersed particles that are approximately two orders of magnitude smaller than the pore openings. It is further assumed that the concentration of the suspended particles applied to the filter remains constant. At present, a theory that will permit departures from these simple assumptions is not well developed.

Kinetics of Filtration

When water containing suspended particles is passed through a clean, granular, porous medium (such as a sand filter), the initial rate of particle removal with depth is proportional to the concentration of particles in the flow. But as removed suspended material accumulates in the pores of the filter, the filtration characteristic alters because of the change in the internal geometry of the porous medium. This change at first increases the rate of removal of suspended material from the flow, but later, as the pores become more and more blocked, the removal rate decreases and ultimately drops to zero when the filter is completely ineffective. As the amount of deposited material in the filter varies with depth as well as with time, it follows that the concentration of material in the flow will also vary with depth and time. If the filter is of a given depth, the time pattern of the change of concentration in the flow at this depth is very important. The water flowing out at this depth is the filtrate, the quality of which is of prime significance to the filter operator.

Mathematics of Filtration

The kinetics of rapid filtration of water containing suspended solids have recently been expressed mathematically as:

$$-\frac{\partial I}{\partial L} = \left(\lambda_0 + c\sigma - \frac{\phi\sigma^2}{f_0 - \sigma}\right)I \dots (1)$$

in which I is the concentration of suspended particles in the flow, and L is

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the distance measured from the filter surface. The parenthetical term in the equation is the rate factor group modifying the concentration I with depth L. Initially (in time), this group contains λ_0, the initial rate factor only, as σ (the filter porosity f_0 enters the expression Thus, Eq 1 is a statement of the dependence of the depth rate of removal of suspended particles on the initial removal characteristic of the filter (λ_0), its subsequent modification by accumu-
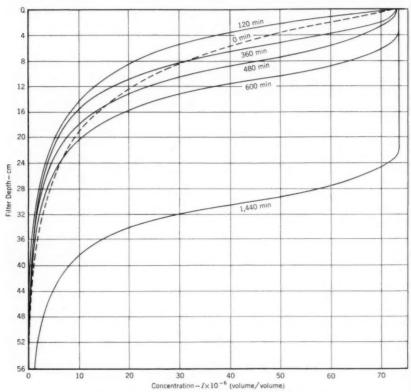
Fig. 1. Variation of Concentration in Flow With Depth

Experimental data: surface concentration I_0, 73×10^6; *geometric mean diameter of sand, 0.0544 cm;* θ, *22.5°C;* v, *8.2 cm/min;* f_0, *0.390;* λ_0, *0.103/cm;* c, *12.1/cm; and* ϕ, *217/cm.*

ratio of the volume of accumulated deposits to unit filter volume) is equal to zero. As time progresses, σ increases, and the rate factor is modified. This is expressed by the rate factor parameters c and ϕ, and the initial

lating deposits (σ), and on the concentration (I). The arguments and supporting evidence for the validity of Eq 1 are given in another article.[1]

In addition, the equation of continuity relates the removal of suspended

material from the flow to the increase of deposits in the pores:

$$\frac{v(\partial I)}{\partial L} + \frac{\partial \sigma}{\partial t} = 0 \ldots\ldots\ldots(2)$$

in which v is the approach velocity of the water to the filter surface, and t is the time. Eq 2 states that the material removed from the flow equals that deposited in the pores. If I and σ are expressed as volumetric concentrations, the equations are dimensionally consistent.

By a combination of Eq 1 and Eq 2 and differentiation, an equation can be produced to express the space-time distribution of material in the filter:

$$\frac{\partial^2 \sigma}{\partial t \cdot \partial L} - \left[\frac{(c+\phi)(f_0-\sigma)^2 - \phi f_0^2}{(\lambda_0 + c\sigma)(f_0-\sigma)^2 - \phi\sigma^2(f_0-\sigma)} \right]$$
$$\times \left(\frac{\partial \sigma}{\partial t} \cdot \frac{\partial \sigma}{\partial L} \right) + \left(\lambda_0 + c\sigma - \frac{\phi\sigma^2}{f_0-\sigma} \right) \frac{\partial \sigma}{\partial t} = 0 \ldots(3)$$

A similar second-order partial differential equation in I can be produced, but it is more complex, containing a polynomial in $\frac{\partial I}{\partial L}$. Because of the complexities involved, there are no known analytic solutions to Eq 3 in σ and its counterpart in I. Consequently, a numerical solution of the filtration equations has been carried out, the iterations of the computational procedure simulating the filtration action.

Filtration Simulation and Computational Block

Before filtration can be simulated by computation, boundary conditions must be known. These are:

$$I = I_0 \quad \text{at} \quad L = 0$$

in which I_0 is the concentration in the inflowing water;

$$\sigma = 0 \quad \text{at} \quad t = 0$$

which indicates that there is no deposit in the filter at the beginning of the filtration; and:

$$I = I_0 e^{-\lambda_0 L} \quad \text{at} \quad t = 0$$

which follows from the integration of Eq 1 when σ equals zero. (The letter e represents the base of the natural system of logarithms, 2.7182.)

As the filtration is one-dimensional in space—that is, throughout the filter depth—this dimension can be divided into equal elements δL, so that at time $t = 0$, the pattern of flow concentration will be as given in the first column of Table 1. This is also shown graphically as the negative exponential curve labeled "0 min" in Fig. 1. As this is a boundary condition, the concentration of deposited material will be zero.

TABLE 1

Variation of Concentration With Depth Increments δL and Time Increments δt

	$t=0$	$t=\delta t$	$t=2\delta t$	$t=3\delta t$
$\delta L \begin{cases} \\ \\ \end{cases}$	I_0 $\downarrow \sigma_0 = 0$ $I_{10} = I_0 e^{-\lambda_0 \delta L}$	I_0 $\downarrow \sigma_{11}$ I_{11}	I_0 $\downarrow \sigma_{12}$ I_{12}	I_0 $\downarrow \sigma_{13}$ I_{13}
$\delta L \begin{cases} \\ \\ \end{cases}$	$\downarrow \sigma_0 = 0$ $I_{20} = I_0 e^{-2\lambda_0 \delta L}$	$\downarrow \sigma_{21}$ I_{21}	$\downarrow \sigma_{22}$ I_{22}	$\downarrow \sigma_{23}$ I_{23}
$\delta L \begin{cases} \\ \\ \end{cases}$	$\downarrow \sigma_0 = 0$ $I_{30} = I_0 e^{-3\lambda_0 \delta L}$	$\downarrow \sigma_{31}$ I_{31}	$\downarrow \sigma_{32}$ I_{32}	$\downarrow \sigma_{33}$ I_{33}
$\downarrow$		$\downarrow$	$\downarrow$	$\downarrow$

After time δt has elapsed, although the surface concentration I_0 has not changed, some deposit will have accumulated in each δL, modifying the purely exponential form of the flow concentration distribution. This is shown in the second column of Table 1, and by the curves of increasing time in Fig. 1. As time increases by increments of δt, the process is reiterated in the third and fourth columns of Table 1, which is a block of computational iterations.

Difference Equations

The values of I_{11}, I_{21}, and I_{31} and σ_{11}, σ_{21}, and σ_{31}, and so on in the pro-

gression, are governed by the coupled differential equations (Eq 1 and 2). These can be expressed as difference equations for determining the numerical values of σ_{11} and I_{11} after time δt, through a depth δL. The average (in time) inflow concentration to the first δL layer is $(I_0 + I_0) \div 2$, and the average outflow concentration is $(I_{10} + I_{11}) \div 2$. The change in concentration of particles deposited in the filter is $(\sigma_{11} - \sigma_0)$.

When Eq 1 and 2 are combined, the result is:

$$\frac{1}{v} \cdot \frac{\partial \sigma}{\partial t} = \left(\lambda_0 + c\sigma - \frac{\phi \sigma^2}{f_0 - \sigma} \right) I \dots (4)$$

In difference form, this becomes:

$$\frac{1(\sigma_{11} - \sigma_0)}{v \delta t}$$
$$= \left(\lambda_0 + c\sigma_{11} - \frac{\phi \sigma_{11}^2}{f_0 - \sigma_{11}} \right) \frac{(I_0 + I_0)}{2}.$$

After rearrangement, collection of terms, and replacement of $(I_0 + I_0) \div 2$ with $\bar{I}$, the result is:

$$[(c + \phi) \bar{I} v \delta t - 1] \sigma_{11}^2$$
$$+ [\sigma_0 + f_0 + (\lambda_0 - cf_0) \bar{I} v \delta t] \sigma_{11}$$
$$- [f_0 \sigma_0 + \lambda_0 f_0 \bar{I} v \delta t] = 0 \dots (5)$$

When this is put in the form

$$A \sigma_{11}^2 + B \sigma_{11} - C = 0$$

then the standard quadratic solution applies:

$$\sigma_{11} = \frac{-B \pm \sqrt{B^2 + 4AC}}{2A}.$$

As the constants λ_0, f_0, c, ϕ, and v are defined for the particular filtration situation (they may be determined from an experimental model), and I and σ_0 are known from the boundary condition, it follows that for a specified time increment, δt, the value of the deposit in the filter (σ_{11}) can be calculated. The continuity equation (Eq 2) becomes:

$$\frac{v}{\delta L} \left[\frac{(I_{10} + I_{11})}{2} - \bar{I} \right] + \frac{1}{\delta t} (\sigma_{11} - \sigma_0) = 0.$$

Rearranged to give I_{11}, the equation is:

$$I_{11} = 2\bar{I} - I_{10} - \frac{2\delta L}{v \delta t} (\sigma_{11} - \sigma_0).$$

As $\bar{I}$, I_{10}, and σ_0 are all known boundary values, and σ_{11} has just been calculated, it follows that I_{11} (the concentration flowing out from a layer δL after time δt) can be computed. It now becomes possible to move to the next δL layer and repeat the computation, as previously calculated values define boundary conditions for this layer. This procedure can be repeated throughout the required depth for the interval δt. With this accomplished, the entire process can be reiterated for another interval δt, and so on for the required time span. Such iterations can be readily accomplished on an electronic digital computer.

Programing

After values for the filtration parameters had been determined by experiment, a filter run was simulated with the use of these parameters on a computer * utilizing an automatic programing system.† With this system, the program is written as a series of instructions using mnemonics, under a defined system of rules, to express the variables and operations. The program was arranged to:

1. Reserve enough space in the computer for computed values to be stored before they are written as output

2. Set the computer to read the values for the filtration parameters and the values for δt and δL increments, as well as their limits (input data)

3. Set the computer to write the input data as a check on the operation.

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* IBM Model 704, manufactured by International Business Machines, New York, N.Y.
† Fortran autocoding system, by IBM.

4. Set boundary values for the iterations and arrange the computation to proceed in groups suitable for printing out

5. Calculate the first negative exponential distribution of the flow concentration

6. Advance the calculation by δt and calculate from the difference equations the values for flow and deposit concentration through the filter depth

7. Write out the required values from these computations

8. Iterate the calculation and output writing until the limit for t is reached

9. Write out the number of the run and a comment that the computation is complete.

The program included various operations that tested the computation as it proceeded, and which would stop the program and write a suitable comment if there was any inconsistency. There was also a convergence criterion set in the program for values of σ (the deposit concentration) which would stop the computation if the whole filter depth reached its equilibrium saturation condition, rendering further calculation unnecessary. The complete program is filed at the Massachusetts Institute of Technology Computation Center, Cambridge, Mass., as "N570 Water Filter Studies," and is available from the director.

The autocoded instructions were punched on cards * and fed to the computer, which translated the instructions to machine-operating binary punched cards. The second deck of binary cards, together with the cards containing the input data, actually operated the machine to produce the desired computed results. These results, written on a magnetic tape during the computation, were ultimately printed on

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* IBM Hollerith cards.

standard paper sheets after the computation was completed.

Approximately 40,000 printed values of flow concentration and deposit concentration could be obtained from a run lasting 12 min of machine time. This is equivalent to operating a filter 1 meter deep for 100 hr, and taking readings of flow concentration and deposit concentration for every centimeter of depth, every 30 min.

Typical Data and Results

A series of experiments was made with a laboratory sand filter receiving water containing planktonic algae. From these experiments, it was possible to derive the filtration parameters, and then to simulate the filtration on the computer and compare the computed distributions of deposited particles with the empiric ones. Such a simulated filter run was made with the following data: inflow velocity (v), 8.2 cm/min; initial porosity (f_0), 0.39; rate factor parameters $(\lambda.)$, 0.14/cm; (c), 12.0/cm; (ϕ), 64.0/cm; surface concentration in the flow (I_0), 135 $\times 10^{-6}$ (volume/volume); depth increments (δL), 1.0 cm; maximum depth (L), 56.0 cm; time increments (δt), 10 min; and maximum time (t), 1,440 min. In units normally employed by water engineers, this is the equivalent of a 24-hr filter run on a 22-in.-deep filter, the filtering water containing 135 ppm algal cells at 120 gph/sq ft.

The simulated filter run on the computer took about 5 min of machine time. Approximately 16,250 values of deposit and flow concentrations were produced; abridged results are given in Table 2.

Complex Situations

The basic concepts outlined in the filtration equation (Eq 1) hold for a variety of more complex filtration situations. The problem with such situ-

ations, however, is to define the nature of the variation of the parameters. Three variations of the simple example previously outlined are frequently met in practice. These are:

1. The concentration of suspended particles in the inflow varies with time.

2. The filter medium is not homogeneous (usually the filter is graded hydraulically by backwashing).

3. The suspension is not homogeneous—that is, there are variations in particle type, size, shape, and density.

In ground water infiltration, there may be a two- or three-dimensional flow; and in certain industrial constant-pressure filters, the approach velocity and temperature may not remain constant throughout a filter run. These situations do not, however, usually occur in water filtration plants.

Variations 1 and 2 can be handled with the computer program. Present knowledge is insufficient to deal satisfactorily with Variation 3, nonhomogeneous suspensions.

TABLE 2

Abridged Results of Simulated Filter Run

Time min	Filter Depth—cm					
	5	15	25	35	45	55
	Flow Concentration, $I \times 10^{-6}$					
0	67.0	16.5	4.1	1.0	0.2	0.1
120	30.5	7.0	1.9	0.5	0.1	0.1
240	36.8	6.8	1.9	0.5	0.1	0.1
360	67.2	8.0	2.1	0.6	0.1	0.1
480	120.2	9.8	2.3	0.6	0.2	0.1
600	135.0	12.8	2.7	0.7	0.2	0.1
720	135.0	18.0	3.3	0.8	0.2	0.1
840	135.0	28.1	4.1	0.9	0.2	0.1
960	135.0	52.2	5.2	1.1	0.3	0.1
1,080	135.0	104.6	6.9	1.4	0.3	0.1
1,200	135.0	133.7	9.6	1.8	0.4	0.1
1,320	135.0	135.0	14.0	2.3	0.5	0.1
1,440	135.0	135.0	22.3	3.0	0.6	0.1
	Deposit Concentration, σ					
0	0	0	0	0	0	0
120	0.0085	0.0011	0.0002	0.0001	0.0000	0.0000
240	0.0177	0.0017	0.0003	0.0001	0.0000	0.0000
360	0.0387	0.0025	0.0005	0.0001	0.0000	0.0000
480	0.0632	0.0036	0.0006	0.0001	0.0000	0.0000
600	0.0692	0.0054	0.0009	0.0002	0.0000	0.0000
720	0.0699	0.0086	0.0012	0.0002	0.0001	0.0000
840	0.0700	0.0151	0.0016	0.0003	0.0001	0.0000
960	0.0700	0.0313	0.0023	0.0004	0.0001	0.0000
1,080	0.0700	0.0585	0.0033	0.0006	0.0001	0.0000
1,200	0.0700	0.0686	0.0048	0.0008	0.0002	0.0000
1,320	0.0700	0.0699	0.0075	0.0011	0.0002	0.0000
1,440	0.0700	0.0700	0.0128	0.0015	0.0003	0.0001

If the concentration in the inflow varies with time, let I_s be the concentration at the surface at any time. Then, if I_0 equals I_s ($t = 0$) and $F(t)$ denotes some function of time,

$$I_s = I_0 \times F(t).$$

If, for example, I_s is the concentration of algal cells in a reservoir, subject to an exponential growth phase, then:

$$I_s = I_0 e^{kt} \dots \dots \dots \dots (6)$$

A simple modification of the program will cause the computer to alter the boundary value of I at $L = 0$, in accordance with Eq 6, for each successive iteration.

Where the filter is nonhomogeneous, a grading curve will establish the variation of grain size (D) with depth:

$$D = F(L).$$

If the variations of λ_0, c, and ϕ with D are known (from experiment), and λ_0, c, and ϕ are designated as the values for the top layer of filter grains, then:

$$\lambda_0' = \lambda_0 F'(L)$$
$$c' = c F''(L)$$
$$\phi' = \phi F'''(L).$$

Therefore, Eq 1 becomes:

$$-\frac{\partial I}{\partial L} = \left(\lambda_0 F'(L) + c F''(L)\sigma - \frac{\phi F'''(L)\sigma^2}{f_0 - \sigma} \right) I$$

on the normal assumption that f_0 does not change with depth. The program will have to be modified to cause the computer to vary the values of λ_0, c, and ϕ as L is incremented. This will expand the program and lengthen the running time, but will be quite possible.

Conclusion

The relatively simple numerical procedure and program described are satisfactory for simulating filtration on an electronic digital computer. The program permits extrapolation of experimental filter runs to be made and design tables to be prepared for filter planning and operation. It is believed that this work is the first application of computational technique to filter studies. As more complex filtration situations come under study, this method will provide a basis for the more complex programs that will result.

Summary

When a fluid containing particles in suspension is filtered through a porous granular medium, particles are removed from the flow and deposited in the pores. The concentrations of particles in the flow and in the pores vary with time and position—that is, when the flow is one-dimensional. Such variations can be expressed by two linked partial differential equations, which, expressed in difference form, can be used to provide numerical distributions of the particles. Such distributions were obtained by using an electronic digital computer, with the program arranged so that the computer numerically simulated the filtration action. Results are given from a computer run, the operating data for which was derived from a filtration experiment.

Acknowledgment

Reference

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Theory of Flow in Filter Media

—Douglas Feben—

IN the opening chapter of M. N. Baker's *The Quest for Pure Water,*[1] a Sanskrit document is quoted, the contents of which date back to 2000 BC: "Impure water . . . may be purified by filtration through sand and coarse gravel. . . ." For approximately 4,000 years following, such treatment constituted an art rather than a science. In the foreword to the same book, Abel Wolman states: ". . . from the late 1890's, rule of thumb has been increasingly replaced by the making and application of the results of scientific research. . . ." Rule of thumb has not yet been completely eliminated either in design or in operation, but the current tempo of water treatment research has quickened to the point where it becomes increasingly difficult, if not impossible, to keep abreast of all that is being written relative to water treatment methods, plant design, and operation. If, then, in this article, some statements cannot be definitely related to the results of scientific investigation, the reason is the continued use of rule of thumb or the author's inadequate assimilation of the literature.

The media discussed include sand, anthracite,* and gravel as used in rapid sand gravity filters. The hydraulic

* Anthrafilt, a product of Anthracite Filter Corp., Wilkes-Barre, Pa.

and other physical characteristics of these media will be especially noted.

Filter Media

According to geologists, the earth is in the throes of one of its periodic tectonic upheavals, a term referring to the slow rise of rocks to form mountains. Geologists further state that the earth has undergone several of these cycles before. Between periods of rock rises, there were much longer periods in which the mountains were slowly leveled by weathering and other forces of nature. When rocks and minerals are broken down, the products are classified as gravels, sands, silts, and clays. This classification is made primarily on a size basis, and petrologists have arbitrarily assigned the limiting sizes of 0.05–2 mm to define sand. Material larger than 2 mm is classified as gravel.

Gravel and sand are produced by attrition of large rock fragments as a result of the action of such moving water as tides, waves of seas and lakes, and the flow of rivers. Because attrition is caused by movement, some hydraulic size classification results. Gravel deposits occur where currents are, or were, more active than those causing the deposition of sand. The pebbles of loose gravels are similar to those in conglomerates, which are pebble beds that have become cemented

by calcareous, siliceous, or other kinds of material.

Because sand results from the disintegration of rock, it is reasonable to expect that most of the minerals constituting rock should be found in sand. For several reasons, quartz is the commonest ingredient. First of all, oxygen and silicon are the two most abundant elements in nature. Therefore quartz, which is a native silica (SiO_2), is abundant in rocks. As it is comparatively hard and has little or no cleavage, it is not readily worn down to a fine state. Furthermore, it is almost insoluble and resists decomposition.

The filter medium processed from selected anthracite is a hard, compact coal, with a sharp conchoidal fracture. It was originally thought that anthracite was metamorphosed bituminous coal—that is, coal from which the greater part of the volatile hydrocarbons had been driven off by heat occasioned by pressure of earth movements or due to the proximity of intrusive igneous rocks. Coals known to have been subjected to such conditions, however, have a higher ash content than that of the true anthracites. A later theory suggests that anthracites originated from vegetation that was subjected to more extensive bacterial action before being permanently submerged in water. In processing, a selected coal is washed—to further reduce the ash content—crushed, and screened. The high carbon content of anthracite makes it relatively inert.

Size Distribution Analysis

When the flow of water through a granular bed and the flow of water through a pipe are compared, it is found that the pipe factors governing the latter flow are the internal dimensions and internal surface features. The pipe wall thickness and the material of which it is constructed do not enter into the picture. Similarly, the flow of water through a bed of granular material is governed not by the chemical composition of the grains but by the geometry of the pores. Because grain sizes and shapes influence the shape, size, and extent of the pores, a knowledge of the former is necessary for an understanding of the latter.

Size distribution analysis involves the shaking of a representative sample of material through a set of standard sieves[2, 3]—the larger the number of sieves used, the more complete the analytic results. Practical limitations keep the number of sieves to a usable maximum; this is achieved when the ratio of the sizes of adjacent sieves is approximately equal to $\sqrt[4]{2}$. Regardless of manufacturing quality, sieves need to be calibrated before the results of mechanical analysis can have statistical value. This can be readily accomplished by using a sand sample of well rounded grains. The last grains to pass through a given sieve are retained, counted, and weighed. The average diameter is computed on the basis of the grains' being perfect spheres. If the material is a good quality silica sand of sp gr 2.65, the formula for the computation is

$$D = 9 \sqrt[3]{\frac{W}{N}}$$

in which D is the diameter of the grains in millimeters; W, the weight of the grains counted in grams; and N, the number of grains counted. This method of sieve calibration, originated by Hazen,[4] is considered superior to other methods.

For analytic purposes, a sample of material must be quartered or riffled down to a representative portion of

approximately 100 g in weight. After being shaken through a nest of clean, calibrated sieves, the material retained on each sieve is carefully removed and weighed. If the weight of material on each sieve is plotted against the sieve size retaining it, a curve similar to that in Fig. 1a is obtained, providing that the sample is a natural material. This is a normal frequency distribution curve quite prevalent in nature. Such a curve frequently results from the examination of any large collection of particles, such as the velocity distribution of the molecules in a gas sample or the energy distribution among the stars of the galactic system.

If these same data are processed on an accumulative basis, then the percentage by weight of material passing any given sieve can be plotted against the corresponding sieve size (Fig. 1b). In Fig. 1a, the bulk of the sample creates the marked curvature in the midrange portion of the curve. In Fig. 1b, the curve is nearly straight, with the exception of the smaller portions at the extremities of the range. If the same data plotted in Fig. 1b are plotted on logarithmic probability paper, the curve shown in Fig. 1c is obtained. A straightening of the curve results from the use of two devices. The vertical scale, being logarithmic, compresses values according to magnitude, thus tending to reduce the curvature produced by the higher values. The probability scale on the horizontal axis is compressed in the median range, with linear distances on either side increasing as the probability of such values lessens. In effect, this latter device produces "straightening" by "stretching."

Hazen's concepts of effective size and uniformity coefficient [4] were developed as a result of working with sand for which there was no hydraulic size classification. Under these conditions, he found that the 10 per cent of small grains had the same influence on the rate of flow as the remaining 90 per cent of larger grains. The size separating these two portions was called the effective size. This can also be defined as the grain size that would cause a material to have the transmission capacity it actually has if all its grains were that size. His uniformity coefficient is, as its name implies, a measure of the degree of uniformity. It is the ratio of the size than which 60 per cent of the sample is finer, to the effective size.
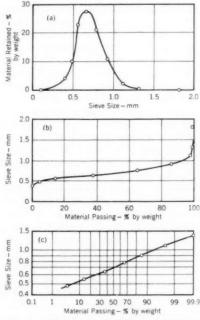
Fig. 1. **Percentage of Materials Passing, and Retained by, Various Sieves**

If the data in Part a are plotted on a cumulative basis, the result is the curve shown in Part b. When the same data were plotted on logarithmic probability paper, the curve in Part c was obtained.

The foregoing definition of the effective size does not always hold true when hydraulic size stratification is produced as in a rapid sand filter. Nevertheless, it has considerable descriptive value today. In graphical analysis, the straight-line equation $y = mx + b$ gives the relationship between the two variables y and x by first locating the point of origin on a vertical axis through the value of the constant b. The slope of the line from this point of origin is then given in the value of the constant m, which is the ratio of the vertical increase to the corresponding horizontal increase. When a size distribution analysis gives a reasonably straight line on logarithmic probability paper, as in Fig. 1c, then the size distribution curve is expressed in the values for its effective size as a point of origin, and the uniformity coefficient as the slope of the curve.

Grain Arrangement and Porosity

The flow of water through a graded bed of sand and gravel is a very complicated process. The details of what possibly takes place can be more readily visualized if the smallest or simplest unit is isolated. For speculative purposes, it can be assumed, first, that the individual grains of sand or gravel are perfect spheres and, second, that the diameter of a sphere in a given group is unity. If the minimum number of spheres in a unit constitutes the simplest configuration, then four spheres can be placed together so that each sphere is in contact with the other three. Their centers then occupy the points of a tetrahedron, the four surfaces of which are equal equilateral triangles.

These four spheres, so placed, do not represent the minimum unit volume of solid and void. This is imme-

diately apparent when an additional single sphere is placed in contact with any three of the original four. The addition of the single sphere produces a second tetrahedron, a surface of which is common to one of the first. In order to obtain the minimum unit volume, one must cut through the spheres to the plane surfaces produced by joining their centers. The unit pore shape available for the passage of water is then revealed by casting a core and removing the sphere sections.

In order for the simplest unit to be acceptable, it must be extensible without limit in every direction. To test this possibility, any two surfaces of the tetrahedron may be selected (Fig. 2). The triangles thus presented will have one common side. On each of these two plane surfaces a perpendicular is erected, originating at a common point on this common base side. The angle subtended by these two perpendiculars will be approximately 70.5 deg. This means that if five equal tetrahedrons are placed together to share a common side, the angles will total approximately 352.5 deg. The difference between this value and 360 deg represents the portion that cannot be filled by extending the number of these tetrahedrons.

This method of using models and casts to demonstrate pore sizes and shapes was effectively used by Slichter in his studies on the motion of underground waters.[5] In this work, he showed that the maximum pore ratio results when the spheres are arranged so that their centers occupy the corners of a cube; such a figure is obviously extensible in every direction. Cutting through the spheres to the plane surfaces joining their centers develops the cube representing the minimum unit volume of solid and void
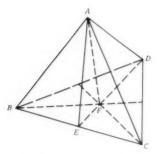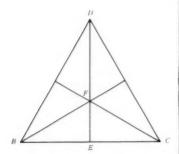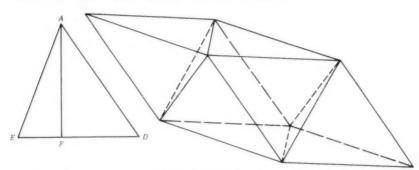
Tetrahedron: four equal equilateral triangles; length of side equals 1. EA and ED are perpendicular to BC. AF is perpendicular to ED and is the height of the tetrahedron.

$$DE = \sqrt{1 - 0.25} = 0.866. \quad FE:EC :: EC:DE. \quad FE:0.5 :: 0.5:0.866. \quad FE = 0.2887.$$

$EA = ED = 0.866.$ $AF = \sqrt{(0.866)^2 - (0.2887)^2} = 0.817$ (height of tetrahedron). Tangent $E = \dfrac{AF}{EF} = \dfrac{0.817}{0.2887} = 2.83.$ Angle $E = 70°32'$.

Rhombohedron: length of side equals 1. Face angles are 60 deg and 120 deg. Area of base is twice the area of Triangle BCD, or $0.5 \times 0.866 \times 2 + 0.866$. Height of rhombohedron is equal to height of tetrahedron, or 0.817. Gross volume is $0.866 \times 0.817 = 0.7075$. Net volume equals sphere volume, or

$$\frac{\pi D^3}{6} = 0.5236.$$

Pore volume equals gross volume minus net volume, or 0.1839. The percentage of porosity is:

$$\frac{100(0.1839)}{0.7075} = 25.99.$$

Cube (developed as explained in text): length of side equals 1; gross volume equals 1. Net volume equals sphere volume, or 0.5236. Pore volume equals gross volume minus net volume, or 0.4764. Percentage of porosity is:

$$\frac{100(0.4764)}{1} = 47.64.$$

Fig. 2. Calculation of Minimum and Maximum Porosities

These calculations are based on the assumptions that individual grains of material are perfect spheres and that the diameter of spheres is unity.

for this geometric arrangement of spheres. The side of the cube is equal to the sphere diameter, and its gross volume is therefore unity. Located at each corner of the cube is one-eighth of a sphere. Therefore, within the cube there is one whole sphere representing the net solid volume which is equal to $\frac{\pi D^3}{6}$. As the diameter D is unity, the net volume is 0.5236. The pore volume is then equal to the difference between the gross cube volume and the net sphere volume, or $1 - 0.5236$. The difference of 0.4764 therefore represents a porosity of 47.64 per cent.

In the same study, Slichter demonstrated that the minimum pore ratio results when the spheres are arranged to occupy the corners of a regular rhombohedron having face angles of 60 and 120 deg (Fig. 2). This figure, like the cube, is extensible in every direction. Development of the minimum unit volume for this arrangement of spheres again shows that the total of all the sphere portions contained within the rhombohedron is equal to one whole sphere. The net solid volume will, therefore, again be 0.5236.

Examination of the rhombohedron reveals that it can be dissected into two tetrahedrons and one octahedron, all having equal surfaces. The tetrahedrons are each separated from opposite and parallel surfaces of the octahedron. The height of the rhombohedron is the height of one of the tetrahedrons. With a sphere diameter of unity, this height is 0.817. The area of one surface of the rhombohedron is 0.866, and its volume is the product of these two values, or 0.7075. The difference between the gross and net volumes is 0.1839, which is the pore volume. Expressed as percentage of porosity, this becomes

$$\frac{100(0.1839)}{0.7075} = 25.99.$$

In comparing the pores with the grains, two important facts should be noted: first, individual pore dimensions are proportional to the dimensions of the grains enclosing the pore; second, the percentage of porosity value is independent of the grain size, but is dependent merely on the manner of packing.

Flow Problems

In a discussion of the relationship between the various factors that govern fluid flow, it is assumed that there will be agreement on terminology and definitions. The value derived from the use of basic formulas and equations is enhanced when one has knowledge of their derivation. Regardless of the extent of one's knowledge of fluid mechanics, a frequent review of fundamentals is often necessary or helpful. Pertinent terminology and definitions are listed in the appendix to this article.

Three separate flow problems occur in the filter bed, and the order in which they occur during an operation cycle is also a convenient order in which to study them.

Initial Head Loss

Initial loss of head is defined as the head required to produce a given flow through a given bed of clean material. To equate the several factors involved, one must first inspect a single pore through which such a flow occurs. The problem then undergoes magnification when such a pore is connected to a string of pores gradually increasing in size.

The laws governing the flow of a fluid through a tube or pipe apply fairly well for other conductors in

which the cross section is not circular. On this basis, there is some justification for attempting a comparison. The pipe formulas in current use are the result of the evolution of empirical laws developed from observations, many of which are as old as the science of hydraulics. Early recognition was given to the following equations, each relating pipe friction loss to the particular factor involved.

1. The friction loss h_f is directly proportional to the area of the wetted surface. If the pipe diameter is D and its length is L, then:

$$h_f = k\pi DL.$$

As both k and π are constants, they can be combined to give:

$$h_f = k_a DL.$$

2. Friction loss varies inversely as some power of the pipe diameter. If the exponent is represented by $m + 1$, then:

$$h_f = \frac{k_b}{D^{m+1}}.$$

3. Friction loss varies directly as some power (n) of the velocity V:

$$h_f = k_c V^n.$$

When these equations are combined, the result is:

$$h_f = \frac{K(DLV^n)}{D^{m+1}} = \frac{K(LV^n)}{D^m}$$

in which K is a coefficient evolved from the various constants in the foregoing equations.

In 1775, Chezy determined that the value of the velocity exponent n was approximately 2. Then, in the nineteenth century, Darcy, Weisbach, and others made further modifications. They proposed a value of 1 for m, and then multiplied and divided by $2g$:

$$h_f = \frac{(2gK)L}{D} \times \frac{V^2}{2g}.$$

With the substitution of the friction factor f for $2gK$, the well known Darcy-Weisbach formula is obtained:

$$h_f = \frac{fL}{D} + \frac{V^2}{2g}.$$

The early use of such formulas was confined to water flow at a limited range of temperatures, in consequence of which the effect of viscosity and
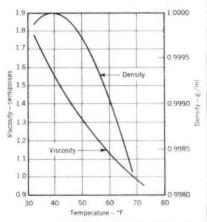
Fig. 3. Relationship Between Temperature, Viscosity, and Density of Water

In the range of $40°-75°F$, the density decreases approximately 0.25 per cent; the viscosity decreases approximately 40 per cent.

density did not assume much importance. It was noted, however, that the velocity–friction loss relationship did not always hold true. This apparent inconsistency was resolved by Osborne Reynolds in 1885, when he demonstrated the existence of two distinct types of flow.

Laminar flow in a pipe is pictured as a number of concentric cylinders,

each of which is sliding along at a slightly higher velocity than that of its outside neighbor. Thus, there is a series of velocities starting with zero for that cylinder of liquid adhering to the pipe wall and reaching a maximum for the innermost cylinder. In such flow, all motion is unidirectional, and there are no cross currents or eddies.

In turbulent flow, motion is no longer unidirectional, except in the overall or total effect. Motion is disorderly, in that at any given point there might be velocity changes in magnitude as well as in direction. Large eddies or whirls develop, break up into smaller eddies, and disappear with the dissipation of energy. The effect of viscosity is manifest in all directions. Turbulence has been cleverly described by L. F. Richardson as that flow in which:

Big whirls have little whirls,
That feed on their velocity;
And little whirls have lesser whirls,
And so on to viscosity.

All other factors remaining constant, laminar flow prevails if the velocity is less than a critical value. Velocity is one of the physical quantities, a combination of which results in a ratio known as Reynolds' number, sometimes designated N_r. Its value is shown as

$$N_r = \frac{\rho V D}{\mu}$$

in which μ is the fluid viscosity; ρ, fluid density; V, fluid velocity; and D, a dimension that, in the instance of flow through a pipe, is the pipe diameter, or, in the instance of a particle surrounded by a fluid, is the particle diameter. For any given conditions, this number is determinate, provided that a consistent set of units is used. Laminar flow exists when this number is less than approximately 2,000. The values from 2,000 to 3,000 constitute a critical range in which either type of flow can occur. Turbulence enters at the higher values.

The friction factor f in the Darcy-Weisbach formula has been shown to be inversely proportional to Reynolds' number. Then, by substitution,

$$h_f = \frac{k_d}{N_r} \times \frac{L}{D} \times \frac{V^2}{2g}.$$

When the constants k_d and $2g$ are combined as the constant C, and the component parts of N_r are substituted, the result is:

$$h_f = C \times \frac{\mu}{\rho V D} \times \frac{L}{D} \times \frac{V^2}{1}$$

$$h_f = C \times \frac{VL}{D^2} \times \frac{\mu}{\rho}.$$

The ratio of the viscosity μ to the density ρ is known as the kinematic viscosity. This value is significant in fluids in which there is a significant variation in both viscosity and density. Figure 3 depicts the changes in these values for water within the temperature ranges encountered in treatment plant operation. In the range of 40°–75°F, the density decreases approximately 0.25 per cent, but the viscosity decreases approximately 40 per cent. The density factor, having a negligible effect, can be dropped from the equation.

It now remains to be seen if the terms in the last equation can be applied to a bed of granular material. The pipe length L can readily be replaced by the depth of the bed as representing the length of a string of connected pores.

The velocity V becomes the interstitial velocity—that is, the average velocity of the water within the filter

bed. When water flows through a clean bed, the interstitial velocity is directly proportional to the approach velocity. This latter value can be substituted, provided that the true relationship between the two velocities is incorporated in the formula. A sphere presents the minimum surface area per unit volume. As the grain shape changes from true sphericity, its surface area per unit volume increases, and porosity likewise increases. With a constant approach velocity, the interstitial velocity will vary inversely with the porosity, according to the equation:

$$V_i = \frac{100 V_a}{P}$$

in which V_i is the interstitial velocity; V_a, the approach velocity; and P, the percentage of porosity. Incorporation of a porosity factor into the formula therefore corrects for differences between approach and interstitial velocities and for any other effects due to variability in grain shape.

The introduction of the porosity variable necessitates a determination of porosity.[3,6] This is readily obtained by preparing a representative sample that, for sand, should weigh approximately 150 g. After it is weighed, the clean, air-free sample is allowed to drop through a column of water filling a Jackson turbidimeter tube. Hydraulic size grading takes place, and the volume of sand is read immediately. If the sand has a sp gr of 2.65, its percentage of porosity P is calculated from its weight W in grams and its volume V in milliliters by the formula

$$P = 100 - \frac{37.7 W}{V}.$$

For material having a specific gravity other than 2.65, the formula is:

$$P = 100 \left(1 - \frac{W}{SV} \right)$$

in which S is the specific gravity of the material.

A porosity value obtained by this method will not duplicate the value obtained from a full-scale filter. There is more arching of the grains in the glass tube than occurs within the filter bed. Furthermore, when water is first permitted to flow through the filter after backwashing, a certain amount of packing takes place, thereby reducing porosity. The amount of packing will depend on grain shape, the rate of flow, and the time taken to reach the required rate of flow. But porosity values so obtained can be used satisfactorily for computation.

The viscosity value can either be used directly, or it can be introduced through its inverse proportionality to temperature. If the latter alternative is used, it should be noted that this inverse proportionality is not a straight-line function, and appropriate correction must be made.

The pipe diameter D can be replaced by the grain diameter, as it has already been shown that the latter is proportional to pore dimension. Grain diameter can only be used if its value remains relatively constant, which does not occur except in beds of extreme uniformity. But because the bed undergoes some degree of hydraulic grading, a formula can be applied to each layer in which size variation is not significant.

A convenient arbitrary selection of layers is that obtained by sieve analysis. If a sufficient number of sieves is used, the material retained on any sieve will have extreme uniformity. Any given portion can be assigned a size value equal to the median size of the two adjacent sieves that separate that portion.[7,8] Then, on the assumption that the same geometric arrangement of grains prevails regardless of

grain size, the percentage by depth of any layer will be equal to its equivalent percentage by weight retained on a sieve. Application of such a formula can be simplified by the insertion of values for porosity, velocity, and viscosity (or temperature). The formula thus reduced can then be applied to each sieve fraction with its own depth and grain size values. Addition of all the results gives the friction loss through the entire bed of material. In summary, the initial head loss increases with an increase in rate of flow and depth of filtering material, and with a decrease in grain size, porosity, and water temperature.

When anthracite is compared with sand, the former will give lower initial head losses than the latter, when all other factors are constant. This is owing largely to the fact that anthracite has porosity values 13–15 per cent greater than that of sand.[9]

Rate of Head Loss

Two theories have been proposed to explain the mechanism of filtration.[10] The interstitial-straining theory emphasizes the variation in width of the opening adjacent to the point of contact of two grains. If the two grains are assumed to be equal spheres, then a line tangential to both grains passes through their point of contact. A second common tangent can be drawn at right angles to the first. The length of this tangent between its points of contact with the spheres is equal to the diameter of the spheres and represents the maximum pore dimension between these spheres. Within the area contained by this tangent and the two arcs joining it to the point of contact of the grains, there is a reduction in width terminating in zero. Therefore, a width of opening exists that will be less than the size of any suspended

particle. The part of the flow that will pass between any two such points will thus be stripped of any particles larger than the distance between these points.

The gravitational sedimentation theory is based on the concept that each pore acts as a chamber in which sedimentation can occur. In a conventional treatment plant, the passage of water vertically downward through the sand bed might be the first time that the direction of flow is parallel to the acceleration of gravity. Because the suspended solids have a density greater than that of water, they will accelerate until equilibrium is established between viscous drag and the force of gravity. If, under these conditions, the particle comes in contact with the solid surface of a grain, it does so with a velocity greater than that of the water. Then, if it is retained by adhesion to the solid surface, it does not reenter the fluid flow.

It can thus be seen that the flow of clear water through a granular medium, complex as it is, is relatively simple when compared to the mechanics of suspended-solids removal in the same medium. If two measurements of head loss in a filter are separated in time by the value t, then the difference in these head loss values divided by t represents the head loss increase per unit time, or the rate of head loss. The duration of a filter run is the criterion by which many operators judge filter performance; this variable is a function of the average rate of head loss.

Two major factors influence the rate of head loss. First, rate of head loss will be proportional to the intensity of surface loading—that is, proportional to the rate at which a unit quantity of suspended solids is brought to a unit surface area of the filter bed. Second, rate of head loss will be in-

versely proportional to that depth of the filter bed which undergoes reduction in permeability as a result of the retention of suspended solids. This latter value is commonly referred to as depth of penetration.

The depth to which a solid particle will ultimately penetrate is the product of two phenomena. The first is the initial entrapment of the particle, which can result from either of the mechanisms described. After this initial retention, solids can penetrate still further into the bed as a result of "creep" or semifluid flow. This in turn will result from the force of gravity reinforced by viscous drag as the interstitial velocities increase because of reduction in pore size.

The fact that rate of head loss is inversely proportional to depth of penetration is apparent when the effects of solids retention are considered. As solid material continues to be retained, a continuous reduction occurs in total pore volume and also in individual pore sizes. Reduction in pore volume means an increase in interstitial velocity, and head loss is proportional to velocity. The foregoing discussion of the geometry of pores in relation to that of grains showed that individual pore dimensions are proportional to the dimensions of the grains inclosing the pores. Reduction in pore size because of clogging therefore produces the effect of a decrease in grain size, and head loss is inversely proportional to the square of this dimension. Owing to size classification, poor penetration concentrates solid matter where its effects are disproportionately greater than when the same amount of material is contained within a larger volume of the bed.

Depth of penetration, together with the duration of filter run, is a function of the following partial list of variables:

1. Filter medium variables
   a. Grain size
   b. Uniformity
   c. Porosity
2. Applied water variables
   a. Concentration of suspended solids
   b. Chemical composition of suspended solids
   c. Physical condition of suspended solids
   d. Concentration of electrolytes
   e. Temperature
   f. pH
3. Operational variables
   a. Rate of filtration
   b. Final loss of head.

The large number of variables involved and the limited amount of knowledge concerning their effects, individually and collectively, have precluded the development of any formula by which filter performance can be accurately predicted. Recognition of this state of affairs has been partly responsible for the creation of AWWA Task Group 2720 P—Filtrability Index Test, which has been charged with the responsibility of investigating the possibility of developing a measurement of filtrability. Until such an index is available, evaluations must be made of the work of a large number of individual investigators. It should be recognized that these published results are acceptable only to the limits within which each investigator worked. Examples of the results of such research merit attention:

Depth of penetration increases as floc concentration, rate of flow, sand size, and concentration of electrolytes increase. Depth of penetration also increases with the water viscosity and with the porosity of the filtering medium. Depth of penetration decreases as the floc is increased in size, as the floc is subjected to aging, and as a

pH of 7.0 is approached.[11] Rate of head loss is directly proportional to the rate of flow.[12] The duration of a filter run varies directly with the square of the grain size and the fourth power of the porosity, and inversely

TABLE 1

Comparison of Sand Specifications, 1932 and 1959

Year*	Depth in.	Effective Size	Uniformity Coefficient
	Minimum		
1932	18	0.33	1.20
1959	20	0.43	1.30
Percentage difference	+11	+30	+8
	Maximum		
1932	37	0.62	2.00
1959	30	0.65	1.75
Percentage difference	−19	+5	−13
	Average		
1932	28	0.44	1.53
1959	26	0.54	1.45
Percentage difference	−7	+23	−5
	Median		
1932	30	0.42	1.60
1959	27	0.54	1.50
Percentage difference	−10	+29	−6

* Values for 1932 are derived from tabulations made by E. A. Hardin[14] of plants in American and Canadian cities of more than 100,000 population. Values for 1959 are derived from tabulations made by the Michigan State Department of Health of 1949–58 data.

with the $\frac{3}{2}$ power of the rate of filtration.[13]

The lack of precise formulation is responsible for the rule of thumb that persists to a large extent in specifying filter media. A trend toward the use of coarser material—a trend resulting

from the many improvements that have been made in plant design and methods for preparing an applied water— is indicated in Table 1. In this table, the 1932 values are taken from data obtained from more than 60 cities in the United States and Canada.[14] Time limitations did not permit securing recent data from so large an area. The 1959 values are those of 21 Michigan installations built since the end of World War II. In spite of the numerical differences, it is believed that the comparison is sound. Table 2 is a breakdown of the 1959 values in Table 1, and indicates the trend resulting from a comparison of pretreatment methods.

In addition to the marked trend toward coarser material, there is also a lesser trend toward greater uniformity. The Committee on Filtering Materials of the ASCE Sanitary Engineering Division reported that uniform sands are as good as, or better than, graded sands and that grading, therefore, does not appear to be a necessary feature. Any departure from complete uniformity involves some degree of packing, with consequent reduction in porosity.

When a choice is to be made between the use of sand and anthracite, the decision should first be influenced by the character of the suspended solids in the applied water. If the solid particles are very small, interstitial straining must be relied on for their removal. In the preceding discussion of this theory, it was demonstrated that when two grains are in contact, there is a width of opening that will be less than the size of any suspended particle. There is a difference, however, between the straining effectiveness of round grains and that of angular grains. The angle subtended from the point of contact by the

surfaces of two angular grains can readily be obtuse, right up to the point of contact. With round grains, the angle becomes more acute as the point of contact is approached and actually terminates in zero. In this respect, round grains are preferred for the removal of finer material.

In some installations, a combination of the two media has been used. When this is done, the water first passes through a predetermined depth

TABLE 2

*Comparison of Sand Specifications by Method of Pretreatment**

Pretreatment†	Depth in.	Effective Size	Uniformity Coefficient
	Minimum		
Conventional	20	0.43	1.30
Upflow	24	0.45	1.30
	Maximum		
Conventional	30	0.60	1.70
Upflow	30	0.65	1.75
	Average		
Conventional	26	0.52	1.44
Upflow	27	0.56	1.47

* Breakdown of 1959 values shown in Table 1.
† Upflow refers to solids-contact process.

of relatively coarse anthracite supported by a lesser depth of finer sand. The high-porosity anthracite permits the deep penetration necessary for low rate of head loss, and the fine sand removes the finer suspended matter that might be passed by the anthracite. This is, in effect, a roughing filter and a polisher all in one unit. When such a combination is used, sizes of material must be carefully chosen in order to maintain the segregation of the two media after backwashing.

Backwash

With regard to filter backwash, two problems pertain to the flow reversal operation: (1) determination of head loss through the expanded bed and (2) determination of the degree of expansion for a given velocity. For an understanding of the hydraulics of backwash, a single spherical particle should first be considered. If this particle is suspended at rest, but completely submerged in water, it loses weight in an amount equal to the weight of the water it displaces. If the particle is permitted to fall, it immediately encounters resistance due to the drag force of the liquid. As its velocity increases, the resistance increases. When the drag force equals the submerged weight of the particle, equilibrium is established, acceleration ceases, and the particle continues to fall at its constant terminal velocity.

If the water, instead of being at rest, is caused to flow vertically upward, the particle will remain suspended, with no movement relative to the observer, when the velocity of the water equals the terminal velocity of the particle. Under these conditions, the drag force is again equal to the submerged weight of the particle. The drag force is the resistance due to flow, and is therefore the head loss resulting from the flow around the particle. The head loss is thus equal to the submerged weight of the particle.

These conclusions may be applied to the entire sand bed. When the backwash is started, the bed is at rest, and the interstitial velocity is a function of the approach velocity and the porosity. When the interstitial velocity exceeds the terminal velocity of the grains, expansion takes place. Porosity increases with expansion, and interstitial velocity is reduced. Equilibrium of the expanded bed is established

when the interstitial velocity of the water equals the terminal velocity of the grains. Under these conditions of equilibrium, the head loss through the expanded material is equal to the submerged weight of the material.

If the approach velocity is increased, the equilibrium will be displaced in the direction that tends to undo the effect of the velocity increase stress. The bed will expand further, increasing the porosity until the interstitial velocity of the water again equals the terminal velocity of the grains. Obviously, therefore, the head loss through an expanded bed in equilibrium is independent of the degree of expansion and is always equal to the submerged weight of the expanded material or the head of water equivalent thereto.

If the filter medium subject to expansion has a void volume equal to P per cent of the total, then the solid material is $(100 - P)$ per cent. If the depth of the filter medium is L, then the equivalent solid depth of zero porosity is:

$$\left(\frac{100 - P}{100} \right), \text{ or } L\left(1 - \frac{P}{100}\right).$$

If this material has a specific gravity of S, its submerged specific gravity is $(S - 1)$. Then the head of water equivalent to the head of solids is the product of these two values, or

$$L\left(1 - \frac{P}{100}\right)(S - 1).$$

This is the friction loss through the expanded portion of the filter.

The relationship between the various factors governing head loss due to flow through a granular medium is independent of the direction of flow. A formula for initial head loss should therefore be applicable to the conditions prevailing during the backwash. It has been shown that the head loss

through the expanded portion of a bed is constant and readily calculated. Its value can then be entered into a head loss formula and the equation solved for any one of the other variables. Thus, for a given temperature, the approach velocity necessary to produce a head loss equal to the submerged weight of sand of a given depth and size is the velocity at which that size sand will start to expand. From that point on, the only variables are the porosity of the expanded bed and the approach velocity, one being a function of the other. The porosity of the expanded bed is thus determinate and can be expressed as the expansion in terms of the idle and expanded porosity values.

Let L equal the depth of the idle bed; L_e, the depth of the expanded bed. Let P and P_e equal the respective porosities of the idle and expanded bed. Then the solids volume of the idle bed is:

$$L\left(1 - \frac{P}{100}\right).$$

In terms of the expanded bed, the solids volume is:

$$L_e\left(1 - \frac{P_e}{100}\right).$$

As the solids volume is unchanged, these equations apply:

$$L\left(1 - \frac{P}{100}\right) = L_e\left(1 - \frac{P_e}{100}\right)$$
$$\text{and } L_e = \frac{L(1 - 0.01P)}{(1 - 0.01P_e)}.$$

The expansion is the difference between the expanded and idle depths, which is $L_e - L$. Then, by substitution,

$$\text{Expansion} = \frac{L(1 - 0.01P)}{(1 - 0.01P_e)} - L$$
$$= L\left[\frac{(1 - 0.01P)}{(1 - 0.01P_e)} - 1\right].$$

It should be noted that linear expansion, calculated as in the foregoing, is based on a single sand size. It is then necessary to compute the linear expansion for each sieve fraction, as was done for initial head loss, and obtain the total expansion by addition.[8]

Some earlier investigators of this phase of filter hydraulics determined experimentally that there was an effective size that appeared to govern the expansion.[18] This was the 30 per cent size, or that size than which 30 per cent of the sand was finer. Their formula equates the four variables of percentage expansion, approach velocity, sand (30 per cent) size, and water temperature. The advantage of this formula is that it requires the use of only one grain size. The disadvantage is the limitations, imposed by its empirical derivation, within which it can be successfully used.

When the backwash behavior of sand and anthracite is compared, a marked difference is noted in their relationship to wash water velocity and expansion. This difference results mainly from the difference in the specific gravities of the two materials: sand, sp gr 2.65; anthracite, sp gr approximately 1.60.[9] For example, a round-grained sand of 42 per cent porosity may be assumed. Anthracite averages 14 per cent greater porosities,[9] which would give it a value of approximately 48 per cent. The same depths of each material would then have 58 per cent solids for sand and 52 per cent for anthracite. As their submerged specific gravities are $(S - 1)$, then their submerged-weights ratio is $[58(2.65 - 1)]:[52(1.60 - 1)]$, or sand:anthracite::3:1, approximately. This represents the head loss ratio during expansion for equal depths of the two media. By the same token, the material with the lower specific gravity will have a lower terminal velocity for the same size particle and will, as a result, expand a greater amount for the same backwash velocity.

Gravel

Filter gravel serves a dual purpose. First, it separates the underdrain system from that portion of the bed which does the actual filtering, thereby preventing penetration of the former by the latter. Second, it supplements the underdrain system in dispersing and equalizing the flow of wash water. It can thus be seen that the design of the underdrain system plays a major role in gravel requirements.

The arbitrary size limitation of 2 mm separates sand from gravel. If the top layer of material is specified as passing No. 8 mesh and retained on No. 10 mesh, it will have a size of approximately 2.0–2.4 mm. If it is specified as not less than $\frac{1}{16}$ in. and not greater than $\frac{3}{16}$ in., then the bulk of the material in that layer will be larger than 2 mm, if the size frequency distribution is normal.

Size limitations of the coarsest gravel are dictated by the sizes and spacings of the individual openings or orifices in the underdrain system and by a consideration of Reynolds' number. At one extreme, if the individual orifices in the underdrain system are smaller in size than the sand grains, then the gravel can be completely eliminated. Porous plate underdrains make this possible.

The loss of head through the gravel is negligible, because laminar flow usually prevails. A Reynolds' number of approximately 2,500 is obtained when the values of its determining factors are: gravel porosity, 40 per cent; particle diameter, 3 in.; water tempera-

ture, 68°F; and wash water approach velocity (wash water rise), 31 ipm. With the exception of porosity, a reduction in any of these values will result in a reduction in the value of Reynolds' number.

Because hydraulic grading of the gravel cannot be secured within the filter, it must be installed in separate layers, each of which must be specified as to size and depth. Certain desirable features in these specifications are practically self-evident. In any layer, the minimum particle size should be not less than the maximum particle size of the layer above it. To minimize packing and maintain porosity, the ratio of the minimum size to the maximum size in any given layer should preferably be not greater than 1:2 and, in any event, not greater than 1:3.

The depth of any layer obviously cannot be less than the diameter of the largest particles in that layer and should not be less than twice that particle diameter. A minimum layer thickness should be specified, and its value should be used when it is greater than twice the largest particle diameter in that layer. The AWWA Standard for Filtering Materials [3] gives a minimum layer thickness of 2 in. It should be obvious that as the specified thickness of any layer is reduced, variations in level acquire greater importance, and more time and care are needed to install the layer.[16] It has been suggested [3, 17] that the gravel be graded in accordance with the equation

$$M = K \log D$$

in which M is the depth in inches below the material of a given size; K, a constant whose value should be in the range 10–14; and D, the particle diameter in millimeters. If such a grading plan is followed, the depth of any given layer can be readily calculated.

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APPENDIX

Terminology, Definitions, and Formulas

The terms pertinent to this article are listed with the term mentioned first, then the symbol or formula for its dimensions given in parentheses, and the definition of the term following:

1. Length (L), measure of linear extent

2. Mass (M), measure of matter (quantity)

3. Time (T), measure of duration

4. Area (L^2), measure of surface

5. Volume (L^3), measure of space bounded by surfaces

6. Density (ML^{-3}), mass per unit volume

7. Velocity (LT^{-1}), time rate of motion

8. Acceleration (LT^{-2}), time rate of velocity change

9. Weight, force (MLT^{-2}), product of mass and acceleration

10. Specific weight $(ML^{-2}T^{-2})$, weight per unit volume

11. Specific gravity (dimensionless), ratio of specific weight (or density) to the specific weight (or density) of a standard substance

12. Viscosity, dynamic $(ML^{-1}T^{-1})$, ratio of shearing stress to rate of shearing strain

13. Viscosity, kinematic (L^2T^{-1}), ratio of dynamic viscosity to density

14. Reynolds' number (dimensionless), ratio of the product of density, velocity, and diameter to dynamic viscosity.

Interstitial Velocity

The relationship between quantity, area, and velocity is expressed as:

$$Q = AV$$

in which Q is the quantity per unit time, and A is the cross-sectional area through which the fluid is moving at the velocity V. If the fluid, after moving through an area equal to A_1 with a velocity of V_1, is required to pass through an area of A_2 with a velocity of V_2, then:

$$Q = A_1V_1 = A_2V_2, \text{ or } V_2 = \frac{A_1V_1}{A_2}.$$

The porosity of a granular medium can be expressed in terms of percentage of porosity. This percentage holds true not only for the voids on a volumetric basis but also for the total pore area in any given plane. Before entering the medium, the fluid can be considered as passing through a 100 per cent porosity, which would be A_1. V_1 would be equivalent to the approach velocity V_a. Then A_2 would be equivalent to the percentage of porosity P of the medium, and V_2 would be equivalent to the interstitial velocity V_i. The foregoing equation can then be rewritten as:

Interstitial velocity

$$= \frac{100 \text{ (approach velocity)}}{\text{percentage of porosity}}, \text{ or } V_i = \frac{100V_a}{P}.$$

Flow Formulas

Poiseuille's formula. Poiseuille's formula for flow through a capillary tube of circular section is

$$f = \frac{\pi a^4 p}{8 \mu l}$$

in which f is the discharge in milliliters per second; a, the radius of the tube; p, the difference in pressure; μ, the coefficient of viscosity; and l, the length of the tube.

Hazen's formula. The formula given by Hazen [4] for flow through the unstratified sand is:

$$V = \frac{cd(2h)(0.70 + 0.03t)}{l}$$

in which V is the velocity of the water in meters per day in a solid column of the same area as that of the sand; c, a constant factor experimentally found to be approximately 1,000; d, the effective size of sand grains; h, the loss of head; l, the thickness of sand through which water passes; and t, the temperature in degrees centigrade. The formula $\frac{t°\text{F} + 10}{60}$ may be substituted for the last term if desired.

Slichter's formula. The formula given by Slichter [5] for flow through unstratified sand is:

$$q = \frac{0.2012 (pd^2 s)}{\mu h K}$$

in which q is the rate at which a quantity of water is transmitted by the column of sand in cubic feet per minute; p, the difference in pressure at the ends of the column (head under which flow occurs) in feet of water; s, the cross-sectional area of the sand column in square feet; h, the length of the column in feet; d, the mean diameter of sand grains in millimeters, or the effective size; μ, the coefficient of viscosity,

or the amount of force necessary to maintain unit difference in velocity between two layers of water a unit distance apart; and K, a coefficient of porosity, some values of which are shown in Table 3.

TABLE 3

Values of Coefficients of Porosity (K)

Porosity %	K^{-1}
26	0.001187
30	0.001905
35	0.003163
40	0.004922
45	0.007295

Hulbert and Feben's formula. The formula given by Hulbert and Feben [7] for flow through stratified sand is:

$$l = \frac{27}{10^5}\left[\frac{dr(73 - p)}{s^{1.89}(t + 20.6)} \right]$$

in which l is the loss of head in feet; d, sand depth in inches; r, rate of flow in million gallons per acre per day; p, porosity, the percentage of voids (as determined by Hulbert and Feben [7]); s, sand size in millimeters (50 per cent median sieve size); and t, temperature in degrees Fahrenheit. Loss is computed for each sieve fraction on the basis that the percentage by weight of material between sieves equals the percentage of material depth in the filter bed.

Fair and Hatch's formula. The formula given by Fair and Hatch [8] for flow through stratified sand is:

$$H = \frac{(KLTFv)S^2}{100}\left(\text{sum of } \frac{P}{d^2} \right)$$

in which H is the loss of head of water passing through the filter bed, in terms of the water column; K, the filtration constant (according to tests by the authors, K is equal to $\frac{5}{g}$ for any con-

sistent system of units of measurement); and L, the vertical depth of the filter bed. T is equal to $\frac{m}{r}$, which is equal to the temperature or viscosity-density factor for filtration, as the viscosity, m, and density, r, of the water both vary with the temperature of the water; F is equal to $\frac{(1-f)^2}{f^3}$, which equals the porosity factor; f is the porosity ratio of the filter bed; v is the velocity of approach over the gross area of the bed; S, the sand shape factor, which varies from 6.0 for spherical sand to 7.7 for angular sand; P, the percentage of sand by weight held between adjacent sieves; and d, the geometric mean of the rated sizes of adjacent sieves.

Expansion Formulas

The formula for expansion given by Hulbert and Herring [16] is:

$$r = [1.04 + 0.01(t - 32)](s - 0.17)e$$
$$+ \frac{5.9}{1 - (s - 0.17)} + 0.24s(t - 32) - 7.4$$

in which r is the rate of application of wash water in inches per minute; e, the resultant expansion, expressed as percentage of original depth; t, the temperature of water in degrees Fahrenheit; and s, the diameter of a grain, such that 30 per cent of the sample, by weight, is finer.
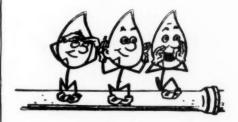
The formulas for expansion given by Fair and Hatch [8] are:

$$E = K_e T_e v \left(\frac{S}{d}\right)^2$$
$$L_e = \frac{L(1-f)}{100}\left(\text{sum of } \frac{P}{1-f_e}\right)$$

in which E equals $\frac{f_e^3}{1-f_e}$, which is the expansion characteristic; f_e is the porosity ratio of the expanded bed; K_e is the expansion constant (according to tests by the authors, K_e equals $\frac{4}{g}$ for any consistent system of units of measurement); T_e equals $\frac{m}{R-r}$, which equals the temperature or viscosity-density factor for expansion; m is the viscosity and r is the density of water. R is a value that lies between 2.60 and 2.65 for quartz sands; V is the velocity of approach over the gross area of the bed; S, the sand shape factor (which varies from 6.0 for spherical sand to 7.7 for angular sand); d, the geometric mean of the rated sizes of adjacent sieves; L_e, the vertical depth of the expanded filter bed; L, the vertical depth of the unexpanded filter bed; f, the porosity ratio of the unexpanded filter bed; f_e, the porosity ratio of the expanded filter bed; and P, the percentage by weight of material held between adjacent sieves.

Percolation and Runoff

Detroit, rather than Bal Harbour, is the focus of our attention now, as we look forward to AWWA's 80th Birthday Conference there Jun. 4–9, 1961. But we aren't likely to forget Bal Harbour soon—its 3,036 relaxed, sportshirted, swimsuited registrants; its spiritedly discussful technical programs, committee meetings, and bull sessions; its wisherama of equipment, materials, and ideas in the carpeted comfort of the Americana ballroom; its starlit evenings of entertainment; its luxurious quarters for everyone and everything; its ocean; its sun; its general aura of wonderland. Bal Harbour was a happy place—happy for work and happy for play, happy for 2,157 men and happy for a record 879 ladies, and happy for AWWA, too, as a great deal was accomplished.

A general review of events and eventists has already appeared in *Willing Water,* and the JOURNAL text pages will carry the full report of technical meetings throughout the year. Caesar and Susie Wertz have found time to sit down again. Ruth Jordan has already spent the $82 that Ex-Secy Harry won in making the closest guess as to the number of AWWA members at the time of the meeting and Helen Stewart, the $83 that WSWMA Manager John earned with the closest estimate of registration. The Americana pool is being used strictly for recreation again. All 1960 medals are pinned, certificates framed, and plaques hung. Miss AWWA 1960 has put away her bathing suit. Even AWWA's trunks are unpacked.

Motor City, here we come, if only in first at first.

Ya gotta sell—the universal slogan of business—certainly points out the means to AWWA's end, too. "Improved water service" won't be achieved until the customer is sold on its desirability, if not necessity. Of course, the Association's Advancement Program is aware of this and is endeavoring to encourage widespread campaigns that will inform the public of the importance of water supply. Basically, though, its efforts are being exerted on a high-level institutional plane and we're beginning to wonder if there'll be any institution left by the time it is sold. It was only two months ago that *Newsweek* pointed out in an article on obesity that "through the stimulation of advertising, tap water is being replaced by sugar juices, milk, and carbonated drinks" in the diet of our teenagers. Meanwhile, news re-

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lease after news release informs us how our industrial customers are succeeding in using less and less of our product. And last month—certainly an indication that the end is in sight—it was announced that "aspirin is now available in suppository form for rectal administration."

We gotta sell. After all, we have naturally what No-Cal must fabricate.

Presidential Twain Meeting: Present Pres. C. F. Wertz (left) presents a Florida palm to handsome Californian, Past Pres. Lauren Grayson. Palms in background are courtesy of the Americana Hotel, headquarters for the AWWA meeting.

Not just no calories, either, but no almost everything unwanted. "For Health and Svelte, Drink Water!" "For Wealth," too, at the moment, but we'd like to change that. "For That Clean Look, Use Water!" Too many women these days are cleansing instead of washing. "Drink Water—No Hangover, No Bottles to Return." And just like vodka, no taste and no breath. "Try Aqua Car Wash," available from your local water utility—apply it with a garden hose, will not damage paint. "Use the Universal Detergent"—no mixing required, will not hurt your hands. "Free to every regular customer—DOUSE—the best fire extinguishing agent available." And, of course, "Suppliers of Eau Johnny—the original saniflush!"

If ya gotta, ya gotta!

Fluorinews still arrives by the bundle, but there appears to be less and less "new" in it. As a matter of fact, it appears to be less and less a matter of fact. Yet some of the items are of interest:

● From Oroville, Calif., whose supply, provided by the California Water Service Co., was fluoridated beginning 3 years ago by order of the state public utilities commission, it is indicated that antifluoridationists are still carrying on the "struggle," debating even the utility's reported cost figures as being only a sixth of what the "real cost" must be and revealing such interesting information as the "fact" that fluorides are manufactured by only one firm in this country.

● From New York City comes not word, but words and words, none of them very new. But Water Commissioner Armand D'Angelo, having studied the problem involved in applying fluorides to the city's vast system, did report that fluoridation is feasible. Health Commissioner Leona Baumgartner supports the practice, as does Mayor Wagner, but as a majority of the city's Board of Estimate apparently does not, the issue will probably not come to a vote.

● From Spartanburg, S.C., comes a report that a county court judge has

restrained the water works commission from fluoridating the supply on the familiar grounds of constitutionality, health, and religion.

● From Savannah, Ga., a review of a sidewalk poll on the subject, although it favored fluoridation 15 to 6, turned up one revealing reason for a negative vote. That came from a clerk who indicated: "I am against it simply because there has been too much uproar about it."

● From Washington, D.C., a similar view is attributed to Rep. David S. King, Utah Democrat, who, noting the controversy concerning fluoridation, feels that everyone should calm down until an impartial investigation of the subject can be conducted by a nine-member commission of nutritionists, biochemists, and toxicologists to turn up *all* the pertinent facts.

● From the Binghamton, N.Y., *Press* has come an editorial expressing surprise that the antifluoridationists have not considered the implications posed by the use of city water on skating rinks; among a series of other questions and answers in the same vein is this:

QUESTION: Is it true that skating on fluoridated ice tends to cause skin blemishes around the face?

British Delegation En Route to Bal Harbour

Visiting the eastern United States in May, nine British water supply officials inspected utility and public health facilities on a tour arranged by AWWA. Prior to attending the Annual Conference at Bal Harbour, the visitors stopped off in Washington to meet with USPHS officials, as shown above (left to right): Arve H. Dahl, deputy chief, USPHS Div. of Water Supply & Pollution Control; A. E. Fordham, treasurer, British Waterworks Assn.; E. Windle Taylor, director of water examination, Metropolitan Water Board, London; Leonard Millis, secretary, British Waterworks Assn.; I. Y. Glynne, president, British Waterworks Assn.; A. G. McLellan, director and general manager, Sunderland & South Shields Water Co.; R. W. Melvin, general manager, Bristol Waterworks Co.; John W. Knutson, chief dental officer, Office of the Surgeon General, USPHS; J. Cockram, past-president, British Waterworks Assn.; J. H. T. Stilgoe, engineer, Liverpool Corp. Water Committee; and R. G. Allen, director and secretary, Water Research Assn., Surrey, England.

ANSWER: These blemishes are caused by falling on ice, even unfluoridated ice, and the condition even then does not appear on the face.

● From Ottawa, Ont., at the end of last year came word that the federal department of health, based on its further studies at Stratford (naturally fluoridated water), Brantford (artificially fluoridated water), and Sarnia (fluoride-deficient water), had issued a new endorsement of fluoridation and indicated that its 11-year study would be continued.

● From Coulter, Iowa, comes the report of a practically cavityless generation which has been drinking well water with 2.2 ppm fluorides since 1940 when a dry 257-ft well was deepened to 714 ft, tapping the naturally fluoridated supply. Actually 117 other Iowa cities and towns are equally fortunate, but only 39 of the more than 400 other communities are fluoridating.

● From London, England, Dr. W. Stewart Ross, reporting to the American Dental Assn., noted that seven European countries—Belgium, Netherlands, Sweden, Russia, Czechoslovakia, Great Britain, and the German Federal Republic—were already adding fluoride to water supplies. He ascribed the delays in adopting fluoridation to the desire of European countries to conduct their own investigations of the practice. Similarly, Dr. Juan Chaneles of Buenos Aires reported interest in fluoridation throughout South America, with tests being conducted in several small towns in Brazil, Chile, Colombia, Paraguay, and Venezuela.

● But from Australia it is reported that Dr. Philip Sutton of Melbourne University casts doubt upon the actual value of fluorides in reducing children's tooth decay in a book entitled *Fluoridation—Errors and Omissions in Experimental Trials*, based upon a study of the experiments at Newburgh, Grand Rapids, Evanston, and Brantford. Apparently Dr. Sutton found the controls questionable, the sampling procedures irregular, and some of the essential factors assumed rather than determined, so that "more than 11 years after the initiation of the last of these trials, the deficiencies of the controls still remain unrecognized."

● Meanwhile the US Public Health Service indicates that it is conducting a test of 300 home installations of a device produced by Permutit Div. that adds 1 ppm of fluoride to tap water, regardless of flow rate. If the tests are successsful, the device will make available an optional means of fluoridating water in communities not now fluoridating and make fluoridation of individual home and farm supplies possible. The device would be installed on a lease basis for a charge now estimated at approximately $3 per month.

Dentist, anyone?

USGS is planning to conduct an inventory of water use in the United States in 1960, to gather information on increased demands and bring up to date similar inventories made in 1950 and 1955.

Data on water use will be tabulated by source of supply—ground or surface—and by location of use—states and river basins. Information obtained from federal, state, local, and private agencies will be utilized. Although a complete report is not anticipated until 1962, preliminary data will be made available prior to that time.

Jarvis D. Davenport, president, Sturgis Water Works Co., Sturgis, S.D., is the AWWA director from the newly organized South Dakota Section. A member of the Association since 1935, he has also been active in the US Chamber of Commerce, where he served as national director (1953–59). In addition to his present position, he is chairman of the board of the Black Hills Power & Light Co., Rapid City, S.D.

'All the water you need, when and where you need it!' is a slogan we have long tried to sell. We have always considered our arguments in favor of it, and the idea behind it, quite conclusive. Inasmuch as those arguments have been written in P&Rgot, it is possible, of course, that they have been mis- or even ununderstood. Thus, we are happy, with the permission of the British magazine, *Engineering,** to present the idea in the Queen's English:

PLAIN WORDS

Here we are at the end of a dry summer. I wonder how long we shall be able to last out next year before we are told not to water our favorite cucumber. Notice that I say next year: it is not this year that we have to worry about a short-

* Reprinted from Vol. 188, No. 4868 (Aug. 7, 1959).

age. Water gets stored up in some underground carvern or clay basin—the hydrologists probably have some jargon to disguise it—and it takes a year or so for the effect of a dry summer to be felt. This year we are all right; it was wet last year. But next year—there probably won't be any water at all.

Personally, I think we are mentally deranged. Here we are spending tens of millions of pounds on providing for plenty of cheap electricity and coal, yet we aren't spending enough on water, the basic commodity of life, primitive or industrial. What is more, we issue white papers to convince ourselves that everything is well thought out, and by statistics prove that we couldn't possibly be short of water, although there are the plain facts in front of us that we have to lay emergency pipelines to some parts of the country every summer, and that most years were are told to cut down. Of course, goes the official argument, supplies would cost more if we built such large reservoirs that we could outlast even the droughtiest year. Who would mind that? My water bill came to the magnificent sum of one pound, eight shillings, and sixpence for a half-year; I wish my electricity and coal bills were even twice that.

Some reasonable person will probably not only try to convince me that everything is being done to expand the supplies, but will go on to taunt me by adding that you can't just build a reservoir in no time. It takes years to convince old Grandpa down in Much Withering that he must move out of his house, or else wake up one morning to find the water level up to his landing. Well, if that is so, it's a pity that we didn't think of it a few years ago.

Incidentally, I have just learned that the consumption of water per head of the population in this country is 50 gallons a day. The consumption continues to increase, and we ought to be able to satisfy all demands.

CAPRICORN

As a matter of fact

In the billions are the water words these days as the Senate Select Committee on Water Resources continues to make news and estimates: 180 billion dollars is the estimate of required water projects by 1980 to keep pace with growing demand and 600 bgd is the estimate of demand in 1980, compared with only 250 bgd today. In the billions, too, was Edison Electric Institute, with the prediction of 506 bil gal consumptive use by US electric plants in 1980. Total use for the year, more than quadrupling 1959 use, is estimated at 106,409 bil gal, almost 300 bgd. The public water supply demand for 1980 has been estimated at a mere 30 bgd.

Safety gains in the water supply field may be indicated by the improvements shown in the records of AWWA safety award winners. The 1957 records showed an injury frequency rate of 10.7 per million man-hours worked; for 1958 the rate was only 8.86. The injury frequency rate in the mattress and bedspring industry hit 20.1 in the third quarter of 1959 and was 14.9 for the first 9 months of that year.

Engineers Joint Council now has a membership, through its member organizations (including AWWA) of almost 300,000 engineers.
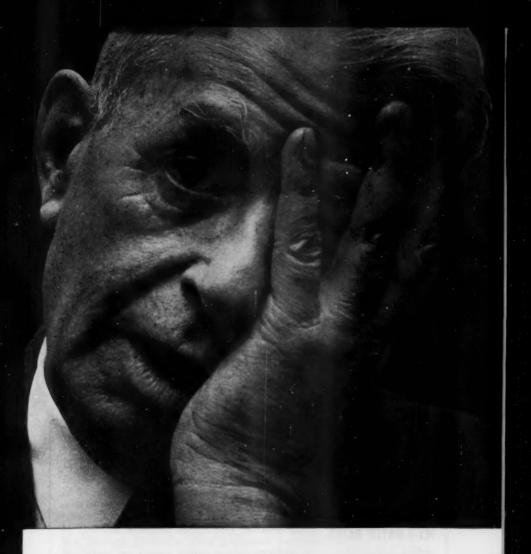
'Water Wonderland,' which Michigan claims on its automobile license plates to be, indicates that the per capita share of surface water for its citizens amounts to 3 acres, at 100 ft average depth, or more than 100 mil gal. With a Great Lakes coastline longer than the eastern seaboard of the United States, more than 11,000 other lakes totaling 727,000 acres, and more than 36,000 miles of natural and artificial watercourses, the state—if not its financial assets—seems to be practically all liquid.

Water system sales—sales of individual home and farm water systems, that is—totaled approximately 800,000 in 1959, according to the report of the Plumbing-Heating-Cooling Information Bureau, Chicago, which noted that the trend toward suburban and exurban living was primarily responsible for the rapid rise from only about 50,000 units per year in 1932.

Meter-reading surveys have indicated that, in the normal, built-up, residential area of a community, a meter reader averages approximately 150 readings per 8-hr day for indoor meters and approximately 225 readings for meters set in outside pits. Missed readings for indoor meters average approximately 15 per cent and for outdoor meters less than 1 per cent. Such factors as spacing of homes, viciousness of dogs, not to mention employee garrulousness, must, of course, be taken into account in any comparison.

"Did I get burned by a low pump bid!"

"Sure we got a low price on our pumps — 3% lower — but we're plenty sorry we didn't write an experience clause into the specs. We've had nothing but trouble and complaints since those pumps went into operation. Sure the manufacturer is cooperative. Why shouldn't he be. He's using *us* as a guinea pig on an untested design. I can't afford to be a testing department for every 'Johnny-come-lately.' I want pumps that *work*. Next time we'll have an 'experience clause.' When it comes to buying pumps ... one burn is too many."

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As a matter of fact

Swimming pool sales in 1959 totaled more than 45,000 and a Gallup poll indicated that swimming was the nation's favorite form of recreation, 33,000,000 adults having participated in it during the year. Meanwhile, International Swimming Pool Corp., of which Esther Williams is president and premier salesman, boosted its sales to $9,000,000 through 762 distributors and went broke, proving no doubt that a good sales figure is not the only requirement of success.

Dropwise is the way Phil Grannan, head of the Corning, N.Y., water department, made headline news last month, giving his customers the inside story on a 3-in. rainfall almost as the last drop fell. Exactly 145,856,493 gal, or 34,192,130,640 cu in., weighing 1,227,516,912 lb, is what the Corning *Leader* quoted him as having counted falling on Corning's 1,817.18 acres. Which would seem to indicate that even in publicity one can make matter of fact.

Rafael V. Urrutia has been appointed executive director of the Puerto Rico Water Resources Authority, which comprises most of the island's electric power system. Since 1954 he has been executive director of the commonwealth's Aqueduct & Sewer Authority. His successor in that position is Juan Labadie Eurite, formerly a government bank executive.

A giant meter, equipped with a column more than 80 ft long, has joined two others in Chicago's municipal water filtration system, as part of a program to increase the plant capacity from 320 mgd to 480 mgd, with a maximum output of 800 mgd. The 36-in. propeller, located 74 ft below sea level, is attached to the meter by an 80-ft drop shaft of 6-in. pipe.

The meter, shown here as it was prepared for shipment, was manufactured by Hersey-Sparling Meter Co. at its El Monte, Calif., plant. Designed to measure the output obtained by the increase in plant capacity, the meter is located at the entrance to several tunnels that transport water throughout Chicago's South Side.

Softener-hardeners will undoubtedly be the next home water service equipment offered if the recent findings of a Dartmouth College physiologist are confirmed. Reporting in the *Journal of the AMA*, Dr. Henry A. Schroeder has noted "a surprising relationship between a high mineral content in drinking water and low death rates from heart and artery disease," thereby seemingly throwing a real blockbuster into the laps of our soft water service friends, not to mention the operators of municipal softening plants. But expansive as some of the health claims, particularly of home water softener agencies, have been, it remains a fact that the real benefits of soft water have nothing to do with drinking it. Thus, we may look forward to the possibility of continuing to use softened hot water for washing, but demanding hardened cold water for drinking.

Correspondence

Doggone Wrong!

To the Editor:

The March issue of the JOURNAL, on p. 48 of Percolation and Runoff, carries a cut with a caption which, in itself, is interesting and amusing.

We presume the caption was repeated from the United Press International item which carried the photo. The caption reads, in part: "which installed a replacement for the hydrant at the left before removing it."

It so happens that the hydrant at the left is one of our Mathews Flanged Barrel hydrants furnished in recent years to the Water Bureau of the Metropolitan District of Hartford and is no doubt the hydrant which was installed to replace the hydrant on the right. We base this on the fact that the hydrant on the right is one which was undoubtedly installed in Hartford many years ago at a time when the water district required all hydrants to be fitted with an outer sliding protection case, with which the hydrant on the

Correspondence (Cont.)

(From page 50 P&R)

right is fitted, but which the district has not required for many years.

This would seem to be a matter of little consequence, except that we do not want other municipalities to suppose that the Hartford, Conn., water utility is abandoning our hydrant. This is contrary to fact.

F. C HARMSTAD

Sales Mgr., Hydrant & Valve Dept., R.D. Wood Co.,
Philadelphia, Apr. 29, 1960.

Friend Harmstad is correct, we derived our caption from UPI's without realizing that it must have been written from the dog's viewpoint. We are happy to note that Hartford is still using Wood hydrants and that in identifying the replacement in the photo, the right wasn't right, it was the left that was to be left.—ED.

Juth Thinking!

To the Editor:

An extension of your "drinking man's filter" idea (November 1959 P&R p. 44) appeared in a recent issue of our local *Kiwanis News*, which reported the comment of a pathologist working on the brain of a lifelong employee of the local filter plant who willed his brain to science. "This is the first time," the pathologist said, "that I've worked on a filtering man's thinker."

I thought you'd enjoy the twist.

JEROME C. ZUFELT

Supt., Board of Water Comrs.,
Sheboygan, Wis.; Apr. 8, 1960

This may be said to ravel the smoke further.—ED.

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TREATMENT—GENERAL

Aeration in Relation to Water Treatment. A. L. DOWNING. *Proc. Soc. Water Treatment & Examination*, 7:2:66 ('58). The term aeration is used to describe processes in which air is brought into intimate contact with a liquid for the purpose of transferring volatile or gaseous substances between the 2 phases. One of the most common applications of the process is in the absorption of atmospheric oxygen in water or aqueous media, either to satisfy the respiratory demands of microorganisms, as in industrial aerobic fermentations or treatment of organic wastes, or to promote chemical oxidns. It is also used as a means of removing or recovering volatile constituents from dilute solns. In this paper some fundamental aspects of the process are examined with particular reference to its use in water treatment. Thus equations are derived relating the performance of aeration units to their size and the rate of flow of water, and the application of these equations to the interpretation of the performance of different types of aerators is discussed. Detailed information on the rates of interchange of oxygen between air and water during aeration is given and approx. predictions of the rates of desorption of undesirable substances, such as hydrogen sulfide, aggressive carbon dioxide, and ammonia, are made.— *PHEA*

Slow Horizontal Filtration in Treatment of Small Quantities of Water. A. M. BERKOV. *Gidrotekhnol. i Melior.* (USSR), 4:20 ('57). A description is given of slow filters with a capacity of 15 cu m/day suitable for use on the farms. The maturation of the filtering membrane occurs in a few hrs and depends on the rapidity of filtration and the turbidity of the filtered water. With a turbidity of water below 200 mg/l, the rapidity of filtration should not exceed 160–170 mm/hr, and with turbidity up to 250 mg/l, 100 mm/hr. On the 1st day of use of filter, the rapidity of filtration should not exceed 50 mm/hr; in the following 2–3 days it should reach the standard rate. The filtered water is quite suitable for use by domestic animals, without addnl. treatment. When used for drinking purposes by the pop., it should be chlorinated.—*PHEA*

Ozonization of Lake Constance Water at the St. Gallen Water Treatment Plant. E. HOFMANN. *Gas- u. Wasserfach.*, 99: 418 ('58). The treatment of Lake Constance water by slow and rapid filtering, as well as chlorine gas sterilization, has always worked well during the warmer parts of the year. Recently taste and odor troubles developed, as a result of phenols in the supply on the south side of the lake. The structural features and assemblage of apparatus according to the M. P. Otto system show 2 symmetrically arranged ozone groups interrupted by a triple enclosed addn. of rapid filter equip. The operation of both ozone groups in addn. to apparatus for handling of air and rapid filtration equip. is automatic and controlled by the intake of raw water. A large flow chart of the system is presented. Results of the method of operation after placing the plant in service in Feb. '57 are shown in accompanying tables. The ozone expt. required 0.9 g/cu m rapid filtered water and the current consumption for ozonization of the rapid filter totaled 51 Wh/cu m. The cost of the ozone installation amounted to 17.5% of the tot. cost of the recovery and treatment of Lake Constance water.—*PHEA*

The Biological Effect of Chlorine and Its Compounds in the Disinfection of Water. E. NAUMANN. *Tech. l'Eau* (Brussels), 12: 136:17 ('58). The use of chlorine in the disinfection of water and waste waters is discussed with special reference to the fol-

lowing aspects: the conception of the disinfection of water as a biol. process; the morphological and physiol. characteristics of bacteria, viruses, and algae in relation to their destruction by chlorine; the action of various disinfectants on bacteria in aqueous media, including hypochlorous acid, alkaline chlorine compds., compds. formed by chlorine with nitrogen compds., organic chlorine compds. (such as filter to a consistency suitable for use in a lime kiln). With a daily water consumption of 25 mil gal, nearly 19 tons per days of calcium oxide can be reclaimed.—*WPA*

Comparative Sanitary Characteristics of Methods of Disinfection of Drinking Water. E. I. RAKUSHINA. *Uchenye Zapiski Gor'kovsk. Gosudarst. Med. Inst.* (USSR), 3:17 ('57). The results of different methods of water disinfection are described. Evaluation of the methods was based on the bactericidal effect of the disinfecting agent upon the microflora of water, on the study of the survival of *Bacillus coli* after disinfection, and on the investigation of organoleptic and physicochem. properties of water. Disinfectants used were chlorinated lime with different amts. of chlorine, ozone obtained in a special laboratory apparatus, bactericidal irradiation from ultraviolet lamps (mercury quartz), and currents of ultrahigh frequency from 22- and 50-watt generators. Analysis of investigation materials (673 expts.) showed that ozonizing is the most effective and perfect method of disinfection. Disinfection by ultraviolet irradiation also deserves attention. Currents of ultrahigh frequency from generators UHF–22 and UHF–50 had only a slight bactericidal effect, and cannot be recommended.—*PHEA*

Study on the Removal of Mineral Iron and Humic Materials From Water. G. V. BENEDEN. *Bul. Centre Belge Etude et Document. Eaux* (Liege), 43:49 ('59). Tests showed that dissolved mineral iron present in water in the form of ferrous bicarbonate or ferrous sulfate could be removed by aeration followed by filtration through paper, but that humic material and humic iron could not be removed by aeration alone, nor by filtration through a column of iron and copper filings (forming a galvanic couple), either before or after aera-

tion. During passage through this column, the humic material absorbed appreciable amts. of metal, and the iron entrained could not be removed by subsequent aeration. Further tests on filtration through sand and various mixtures showed that, although mineral iron could be removed by sand or carbon alone, humic materials and humic iron could only be removed by mixtures (the best results being obtained with a clay-sand-ochre mixture) and the columns soon became saturated with humic materials. Electrolysis (with the electrodes at the top and bottom of the filter column) was found to improve the retention of humic material and humic iron on the column. Electrolysis expts. to det. the conditions giving most complete removal of iron and humic material showed that the best combination of electrodes was an aluminium anode and a carbon cathode. The results, presented graphically and diagrammatically, show that the iron can be almost completely removed from water (by coagulation and deposition on the anode); during the first 20 min. colored humic material migrates to the anode, at first without any iron; during the next 20 min a 2nd fraction of colored humic material migrates together with the iron; and subsequently the remaining uncolored humic material. It is apparent that iron which cannot be removed from water by conventional methods is combined with the colored humic material.—*WPA*

LABORATORY AND FIELD METHODS

A Battery-Operated Water Sampler. G. E. EDEN & K. V. MELBOURNE. *Chem. & Ind.*, 220 ('59). An illustrated description is given of portable equip. to collect water samples from open channels at intervals over prolonged periods, for subsequent analysis of the individual samples. It consists essentially of a number of sample bottles, each of which is evacuated and connected to the water source by a rubber tube. The clamp which holds this tube closed can be released electrically and the water is forced into the bottle by atmospheric pressure. The electromagnetically released latch mechanism and the circuit of the timing unit are described with the aid of diagrams. The equip. can be modified to take samples at regular intervals or according to any sampling program.—*WPA*

An Electrode System for Measuring Dissolved Oxygen. D. E. CARRITT & J. W. KANWISHER. *Anal. Chem.*, 31:5 ('59). In the detn. of dissolved oxygen by polarographic devices with solid electrodes, difficulty is experienced because of electrode poisoning and high temp. coef. A system has now been devised comprising a platinum electrode, a silver–silver oxide reference electrode, a thermistor which provides temp. compensation, 0.5 N potassium hydroxide as an internal electrolyte, and a polyethylene membrane pulled tightly over both electrodes. The membrane is permeable to dissolved molecular oxygen, but much less so to other dissolved substances, and permits the passage of oxygen to the platinum electrode, where it is reduced to hydroxyl, but prevents the passage of substances that would poison or change the characteristics of the electrode. The device has operated satisfactorily in sea water, fresh water, and sewage. Addition to the external environment of chlorine, hydrogen sulfide, acids, and bases had no measurable effect.—*WPA*

Determination of ABS in River Waters and Sewage. H. L. WEBSTER & J. HALLIDAY. *Analyst* (Gr. Br.), 84:552 ('59). An improved colorimetric method based on principles reported in earlier methods is now reported for the detn. of ABS (**I**) in natural and river water. Steps include an acid hydrolysis and removal of hydrolyzable materials, a modified Fairing and Short amine extn. to isolate the detergent from many interfering substances, removal of excess 1-methylheptylamine by boiling with alkali, and colorimetric detn. of **I** with methylene blue, according to the procedure of Longwell and Maniece. Agreement with infrared analysis is good, and the method is of value when infrared equip. is not available or when a more rapid result is required. It can also be satisfactorily applied to sewage effluent and settled sewage. It gives good recoveries of added detergent and is reproducible.—*CA*

Determination of Radioactivity in Saline Water. V. SODD; A. GOLDIN; & R. VELTEN. *Anal. Chem.*, 32:25 ('60). In connection with radioactive-waste disposal it is necessary to det. the background levels and low concn. of radionuclides in sea water prior to the disposal operation and to monitor

during operation. A screening method for gross detn. is based on pptn. of many cations as sulfides from NH_4OH soln. Carrier La, Fe(III), Co(II), Zn, and Ni are pptd. while the bulk sea water salts remain in soln. An alk. earth fraction contg. Sr, Ba, and Ra can be recovered by pptn. of carbonate from the sulfide filtrate followed by pptn. of Sr carrier as nitrate. About 95% of the Cs can be recovered sep. by pptn. by using silico-tungstic acid and conventional purification.—*CA*

Absorptiometric Determination of Aluminum in Water. R. F. PACKHAM. *Proc. Soc. Water Treatment & Examination*, 7:2: 102 ('58). The immediate purpose of this investn. was the selection of a suitable method for the detn. of aluminum required in connection with a long-term research project on the coag. process of water treatment. It also had the more general aim of discovering the best method of aluminum estn. that could be recommended as a water works practice. Absorptiometric methods of colorimetric analysis have many advantages over the visual-comparison techniques. The repeated prepn. of standards is rendered unnecessary, human errors are substantially diminished, and the results obtained are more precise and certain. The present study was therefore confined to those methods which could be used with an absorptiometer. This did not exclude the consideration of visual-comparison techniques which could be adapted for absorptiometric work. The requirements of the eye and absorptiometer are not identical. The eye is very sensitive to slight differences of tint, but considerably less so to the intensity of color. The absorptiometer on the other hand is an instrument designed to measure color intensity. In the detn. of a substance in soln. the absorptiometer reading does not vary directly as the concentration, but bears a more complex relationship to it. This is manifested as a lack of linearity in the calibration plots with a consequent variation in sensitivity, and in order to overcome this it is necessary to calibrate the instrument in detail. In the present work, 4 methods for the colorimetric detn. of aluminum were examined to discover the one which was most suitable. The selected method was then examined in detail to det. the influence of various factors upon its reliability.—*PHEA*

FLUIDICS* AT WORK

Measurement of the Flocculation of Suspensions. R. L. WHITMORE. *Nature* (Lond.), 183:313 ('59). Expts. showed that the degree of coagulation of shale and clay suspensions can be measured by a method used for measuring the degree of coagulation of blood and described by Fahraeus at the Third International Rheological Congress (Ger. '58). The method consists of pressure filtration through a stack of filter papers and measurement of the quantity retained on each filter paper. The greater the degree of coagulation, the smaller is the number of filter papers through which the suspension can penetrate. The method was used for testing a suspension of coal measure shale before and after coagulation with potash alum and the 2 series of filter papers obtained are shown in a figure.—*WPA*

Measurement of the Flocculation of Suspensions. E. R. BRAITHWAITE. *Nature* (Lond.), 183:740 ('59). The author reports that the method described by Whitmore for measuring the coagulation of suspensions (see preceding abstract) has been used successfully for testing "Oildag" (a dispersion of colloidal graphite in oil) and that a special apparatus has been developed which allows a roll of filter paper to be folded into a pack of 25 or more papers, suitable for use in the test. A figure is given showing a typical series of filter papers obtained by this method.—*WPA*
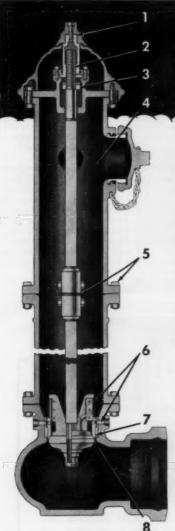
Selecting Tracers for Ground Water Studies. E. HALEVY. *Atomic World*, 10: 184 (May '59). In order to det. movement of water in geological formations and soils an investn. was carried out using radioactive isotopes as ground water tracers. Lab. tests were run using glass columns 150–350 cm long and 10 cm in diam. filled with finely crushed rock of the stratum being investd. (marble-like crystalline limestone). Water at a flow rate of 250 ml/min was passed down the column until, after prolonged washing, a pulse of isotope and carrier liquid totalling 250 ml was inserted. Successive effluent samples of 250 ml were collected and assayed until the activity dropped to background. Tests using iodine–131, chlorine–36, and zinc–65 EDTA on the column showed recoveries of 88%, 99%, and 98% respectively. Iridium–192 was tested,

but it was found that 74% of the EDTA complex was retained in the column. An anionic form of iridium (NH_4)$_2$$IrCl_6$ was tried but was found to deteriorate with time. Only 12% retention of cobalt–60 EDTA was found, so Co^{60} and I^{131} were selected for field trials. In field expts. KI^{131}, Co^{60}-EDTA and K_3Co^{60}(CN)$_6$ along with corresponding carriers were injected into aquifer and subsequently recovered by pumping (a single-well pulse technique). In planning a recharge program of the Yarkon River water in the Lydda well field it was deemed necessary to investigate the nature of the connection between the recharged aquifer and the discharging stream, situated 5 kilometers apart. In prelim. tests pulses of Co^{60}-EDTA and I^{131} were injected into the aquifer. Recoveries were much lower than expected. Due to the failure of Co^{60}-EDTA in the field, an anionic form of cobalt was tested. The choice was $K_3Co(CN)_6$. This is highly stable. Recovery of $Co(CN)_6^{3-}$ after various periods in the aquifer was higher than Co-EDTA or I^{131} and indicates that $Co^{60}(CN)_6^{3-}$ is preferable as a ground water tracer in the Lydda aquifer.—*PHEA*

RADIOACTIVITY

Radioactivity and Water Supply. W. KOECK. *Chem. Ind.*, 9:94 ('57); *Chem. Zentr.* (Ger.), 128:12955 ('57). The author deals with types of radiation, the possibility of radioactive pollution of ground and surface water, and methods of prevention and protection. The water industry must be informed of plans for nuclear-energy plants so that they can influence choice of site and methods of protection.—*WPA*

Future Problems Posed by Radioactive Effluents in Belgium. P. DEJONGHE. *Bul. Centre Belge Etude et Document. Eaux* (Liege), 40:134 ('58). Author discusses the increasing use of radioactive isotopes and development of atomic-power stations in Belgium, with particular reference to the treatment and disposal of contaminated waste waters from labs. and hospitals using radioactive isotopes, from nuclear reactors, and from installations for the reprocessing of uranium fuel elements. The principal aim of the treatment is to concentrate the radioactivity of the effluent into the min. vol.

of sludge; methods for disposal of the resultant sludge are discussed with examples from England and the U.S. In Belgium, sludge has, until now, been stored at the nuclear energy research station at Moll. Possibilities considered for dealing with the increased volumes of sludge in the future include disposal in disused mines, transport to an international disposal ground in a desert region, or dumping in the Atlantic ocean.—*WPA*
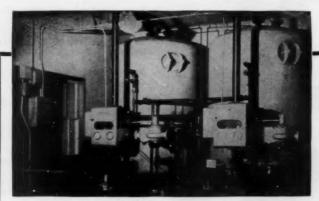
Water Supply Problems in Connection With the Development of Atomic Energy. B. H. Dieterich. *Gas, Wasser, Waerme* (Ger.), 12:265 ('58). The author first deals generally with the risks of pollution of water by radioactivity and the dangers of generalization in the present limited state of knowledge. Precautions required must develop along with the development of the industry. He then discusses the problems of immediate interest to the water supply industry, dealing with the amts. and types of radiation harmful to humans, the radio-

activity existing naturally in the human body and its environment, the introduction of radioactive substances to the human body by means of water and their accumulation and action, calculation of the permissible concns. of radioactive substances in water, the natural and artificial sources of radioactive pollution of water, practical methods of removing radioactivity from water supplies, means of avoiding and reducing radioactive pollution, and the importance of regular measurements of radioactivity at as many water utilities as possible.—*WPA*

Environmental Surveys for Nuclear Facilities. C. A. Pelletier. *Nucleonics*, 17: 56 ('59). The need for environmental surveys in the areas surrounding nuclear facilities is discussed with reference to the sampling of ground water, surface waters (including the water itself, biologic life, and bottom mud and silt), air, and land. The costs of such surveys are considered, including the equipment and staff required.—*WPA*

Radiation and Its Hazards. C. W. Shilling. *Atomics & Nuclear Energy* (London), 9:198 (Jun. '58). The exposures derived from natural radioactivity, x-ray, and radioactive fallout are reviewed. In the case of fallout, we must distinguish between radiation outside the body—that is, external sources, and radiation from sources within the body resulting from the ingestion of radioactive material. Strontium-90, the most important radionuclide produced during an atomic-bomb explosion, may cause leukemia or bone tumors, since its beta rays penetrate through no more than ⅓ in. of tissue, and will have a lesser effect on the gonads. It has been estimated that the gonadal dose is $\frac{1}{70}$ of that to the bone. The author continues with a discussion of whole body irradiation, effect of radiation on genes, the analysis of mutants, and various other factors of general interest.—*PHEA*

Some Comments on Limits and Significance of the 'Permissible Radiation Dose'—I. Analysis of the NCRP recommendation on Radiation Exposure Levels. L. E. Preuss. *Henry Ford Hosp. Med. Bul.*, 5:47 ('57). The executive committee of the National Committee on Radiation Protection recently proposed a series of significant changes which, if accepted, will lower the tolerable radiation exposure limits. These recommendations, which are to be presented to the full National Committee on Radiation Protection, may be summarized as follows: (1) The new system retains the present basic max. of 300 mrem/wk, with a max. of 15 rems/yr (assuming a 2-wk period away from the source of radiation). This also includes the previous extended time penalty unit of 13 wk at 77% tolerance for those instances in which the weekly permissible value was exceeded. (2) One emergency dose of 25 rem is permitted in a lifetime. (3) The max. accumulated dose is not to exceed an avg. of 5 rem/yr. Thus, for the 50 yr of life between ages 20 and 70, a tot. dosage exposure of 250 rem should constitute the max. tolerable accumulation. (4) The committee recommends for the pop. as a whole, a limit of 1.4×10^{2} rem delivered to the reproductive tissues for every 10^{6} persons to age 30; or an avg. of 14 rem per person for the entire pop. through age 30, and one third of this value for each decade thereafter for this pop. unit.—*PHEA*

Radioactivity and the Distribution of Water From Lake Constance. J. Holluta. *Monatsbul. Schweiz. Ver. Gas. u. Wasserfachmaenner*, 38:102 ('58). After a general discussion of the increasing use of radioactive substances and the causes and effects of radioactive contamn. of ground and surface waters, the radioactivity of L. Constance is considered. The radioactivity of water in the lake near Riet, 40 m below the surface, increased tenfold, from 0.02 to 0.22 $\times 10^{-10}$ c/1 in the period May '56 to Aug. '57. In most cases, water taken from the lake for public supplies is only disinfected, and the treatment plants are incapable of reducing the radioactivity of the water. Even when the water is filtered, some substances, such as strontium-90, remain almost unchanged. The treatment of radioactive waste waters is also discussed. It is emphasized that any atomic plants to be built in the area should treat their waste waters to reduce the concn. of radioactive substances below the danger level, and provide auxiliary protection for use in the case of accidents; the storage of radioactive wastes in the basin of L. Constance should be prohibited.—*WPA*

Radioactivity of Surface and Ground Waters in the District of West Berlin. G. Gad & T. Kempe. *Gesundh. Ing.*, (Ger.), 79:84 ('58). In view of the construction of an exptl. reactor on the Wannsee and of the increasing use of active isotopes in hospitals and labs., investns. were made of the present conditions of radioactivity in surface waters and ground waters used for supply in the district of W. Berlin. A map is given showing sampling places for surface waters, which were chosen to include discharges from the reactor and from sewers in various parts of the town. The taking and prepn. of samples and the methods used for the measurement of β- and γ-activity and of the content of potassium are described and the results of three series of measurements are given. In all cases activity was lower than the mpc of 1×10^{-7} μc/ml recommended by the International Radiological Congress in '53. In some cases the value was higher than the safety value of 1×10^{-8} uc/ml recommended for waters in contact with extended areas of pop.; this value was, however, generally caused by increased contents of potassium. No noticeable effect of sew-

age discharges was observed. Some slight variations were ascribed to seasonal variations in flow. The results show that, especially in surface waters, a reliable assessment of total activity is not possible without knowledge of the content of potassium. In order to examine the possibility of accumulation of activity, sludge and plankton samples are also being examined.—WPA

Effects of Uranium Ore Refinery Wastes on Receiving Waters. E. C. Tsivoglou. *Sew. & Ind. Wastes,* 30:1012 ('58). In '55–'56 surveys were made of the condition of streams receiving waste waters from uranium refining operations in the Colorado Plateau area. It was found that the gross radioactivity and concn. of soluble radium in the effluents from a number of the mills and in samples of water from some of the receiving streams exceeded the usually accepted MPC's. All but one of the mills in operation at the time of the surveys discharged most of their wastes to tailing ponds

and at these sites the streams received only small vols. of effluent directly, in addn. to seepage from the tailings pond. At the other mill, the tailings pond was not in use and all waste waters, including sands and slimes, were discharged to the receiving stream. It was found that stream biota and samples of silt were good indicators of recent radioactive pollution, the biota reflecting mainly the concn. of soluble radioactivity in the water while the silt indicates the radioactivity in suspension. In view of the long half-life of radium, the ultimate fate of radium in streams is important, and further studies are required on this subject.—WPA

A Bibliographic Study on Uptake and Accumulation of Nutrient Elements by Plants in View of the Possible Occurrence of Radioactive Isotopes of These Elements Among the Wastes of Nuclear Reactors. E. Wassink; J. Stolwijk; & A. Westerhof. *Meded. Landbouwhogeschool Wageningen* (Neth.), 56:16 ('56). The

accumulation of radioactive materials in plants may cause danger for the consumers of the plants, and a detailed review has therefore been made of literature concerning the possibility of plants accumulating elements from their surroundings, which might be contaminated with waste products from nuclear reactors. The uptake by plants of the various elements from the nutrient medium is affected by several factors, including plant species, stage of growth of the plant, the part of the plant, the acidity and chemical composition of the nutrient medium, temperature, light, and structure and moisture content of the soil. It appeared that elements commonly found in waste waters from nuclear reactors, such as molybdenum, caesium, strontium, iodine, yttrium, and phosphorus, are all accumulated by aquatic plants. Some elements—cerium, niobium, tellurium, yttrium, and zirconium—are accumulated in the root-tips of land plants, and iodine is found in halophytes. Iodine is the most hazardous gaseous waste product. It is concluded that regular tests should be made of the radioactivity of crops in the vicinity of nuclear reactors; that the available phosphorus in soils near reactors should be determined regularly; that special precautions should be taken as regards aquatic plants and plankton that may grow in waste water channels; that the max. permissible concn. of strontium in waste waters should be ten times less than at present, to prevent excessive accumulation; and that the max. permissible concn. of iodine in gaseous wastes should be reduced to 1×10^{-12} μc/cu m. The need for further expts. on this subject is stressed.—*WPA*

Concentration of Cesium-137 by Algae. L. WILLIAMS & H. SWANSON. *Science*, 127: 187 ('58). As cesium-137 is one of the critical fission products in reactor wastes and atomic explosion fallouts, and as it has a fairly long half-life and is water soluble, it may be expected to increase in the environment as more use is made of nuclear energy. Expts. were therefore carried out on the accumulation of cesium-137 by freshwater algae, particularly *Euglena intermedia* and *Chlorella pyrenoidosa*. It was found that both these algae concentrate cesium to a considerable extent. Traces of potassium in the medium appear to have a slight depressant effect on the uptake of cesium. It was also found that dead cells of *Chlorella*

show a high concn. factor for cesium, indicating that structural components persist in the dead cells which are capable of absorbing cesium from very dilute solns. This adsorption is not affected by the presence of potassium.—*WPA*
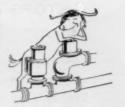
Wastes From the Production of Reactor-Grade Zirconium. R. C. PALANGE ET AL. *Sew. & Ind. Wastes*, 31:565 ('59). The use of zirconium is expanded rapidly as its properties make it a valuable structural material for nuclear reactors. The manufacture of reactor-grade zirconium involves two stages, the sepn. of hafnium from zirconium by chem. means, and the reduction of the purified hafnium and zirconium compds. to pure metal. All the liquid wastes are produced in the first stage, and in the second stage only uncontaminated cooling water is discharged. As no information was available on the waste waters from this industry, a survey was made at one plant to determine the volume, composition and polluting effects of the liquid process wastes and their effect on the receiving stream. Details of the manufacturing processes and the sources of the waste waters are given. The most toxic component of the waste waters was found to be ammonia. The waste waters also contained traces of cyanide, thiocyanates, sulfates, chlorides, and hexone. Zirconium itself does not appear to be particularly toxic. At the plant studied, the waste waters are stored in two lagoons, one for strong process wastes and the other for washing waters, leaks and spills. The wastes from the "strong" lagoon, after storage for several weeks, are carried by truck to a disposal area where adequate dilution is available; the effluent from the other lagoon is discharged at a controlled rate to a small stream. Consideration is now being given to reducing the toxicity of the waste waters by chlorination to destroy ammonia and cyanide or by lime treatment and steam stripping to recover ammonia.—*WPA*

Atomic Energy Program for Waste Disposal. C. P. STRAUB. *Ind. Wastes*, 3:42 ('58). The sources of radioactive wastes in the atomic-energy industry are discussed with reference to gaseous and other airborne wastes, solid wastes (combustible and noncombustible), and liquid wastes (low, intermediate, and high level). Work on the

treatment and disposal of low- and high-level wastes is reviewed.—*WPA*

Ground Disposal of Radioactive Wastes. C. B. AMPHLETT. *Atomics,* 9:194 ('58). The general principles underlying underground disposal of radioactive waste waters are reviewed and examples illustrating the application of this method in the US and Canada are given. Phys. factors to be considered when selecting a site for disposal of radioactive waste waters are outlined and the chem. reactions involved, including adsorption, pptn. of insoluble species, and ion exchange, are described. The methods employed for the disposal of medium- and low-activity waste waters at Hanford, Wash., Oak Ridge, Tenn., Savannah R. site in N. Carolina and Chalk R. site, Ont. are outlined. Serious consideration is also being given to the discharge of highly active processing waste waters and 2 methods, one based on relatively shallow disposal to suitable strata and the other on disposal into deep wells, are outlined. Choice of site and costs of these deep wells are discussed.—*WPA*

Handling Contamination in Spent-Fuel Shield Water. W. SNAVELY & P. MANNO. *Nucleonics,* 17:3:121 ('59). The method employed at the Continental Oil Company refinery, Ponca City, Okla., for treating the 2 radioactive contaminants, strontium-90 and zinc-65, present in the spent material testing reactor fuel shield water, is described. The contamination problem has been reduced by circulating the shield water through a small ion-exchange unit containing Amberlite-120 and then through a larger unit. The small unit is decontaminated frequently, and the radioactivity accumulated in the larger unit is collected every 6 mo., pptd., and disposed of in solid form.—*WPA*

Radioactive Contaminant Removal From Waste Water—Engineering Design Features. M. C. CULBREATH. *Proc. ASCE, J. San. Eng. Div.,* 85:SA3:41 ('59). The first of 2 papers relating to the design and operation of a plant at Oak Ridge National Laboratory on the collection, treatment, and disposal of large volume (0.5–0.7 mgd), low-level radioactive liquid waste. The paper covers the design of the lime-soda softening process treatment plant for the removal of strontium and the tot. rare earths from process waste water.—*PHEA*

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INSTRUMENTATION AND CHLORINATION

Radioactive Contaminant Removal From Waste Water—Evaluation of Performance. K. Cowser & R. Morton. *Proc. ASCE, J. San. Eng. Div.*, 85:SA3:55 ('59). The second of 2 papers relating to the design and operation of a plant at the Oak Ridge National Laboratory for the collection, treatment, and disposal of large volume (0.5–0.7 mgd), low-level radioactive liquid waste. The paper covers the performance of the lime-soda softening process treatment plant and shows that it is an effective and economical method for the removal of strontium and other materials.—*PHEA*

Treatment of Radioactive Waste Waters. P. Dejonghe. *Bul. Mens. Centre Belge Etude et Document. Eaux* (Liege), 73:4 ('57). After a discussion and review of the methods available for the treatment and disposal of radioactive waste waters, of low and high activity, and of radioactive sludges the methods used at the labs. of the *Centre d'Études pour les Applications de l'Energie nucléaire*, at Moll, Belgium, are described. Treatment consists of segregation of waters of low and high activity; sampling to determine the amts. of reagents required; addition of lime and sodium phosphate to remove such ions as strontium and cerium; addition of caustic soda and sedimentation; treatment with ferrous sulfate to remove such ions as ruthenium; a second sedimentation; and filtration through sawdust to remove the remaining radioactivity (cesium and traces of other elements). After sampling to determine the radioactivity, the treated waste water is discharged to the R. Nethe, or, if necessary, retreated. The river water is sampled frequently and the radioactivity of the river is within the limits of tolerance for potable waters.—*WPA*

Gathering and Dispersion of Radioactive Waste. Vincenzo Figura. *Rass. Med. Ind. e Igiene del Lavoro*, 28:50 ('59). The author illustrates all that is known about the gathering and dispersion of radioactive solid and liquid wastes, from reactors and radiochem. industries. He describes the chem. methods of recuperation, for industrial purposes, of cesium-137 and strontium-90, processes based on natural decay, on concn., extraction of radioactive gas, on combustion: he touches upon the collection system of rock-salt in mines, in stainless steel containers, in cement, clay and glass blocks.

He refers to the modalities of interment and unloading in undersea pits and valleys of wastes at high activity, the methods of waste elimination at middle and low activity under water, with particular reference to the research carried out at Windscale and the relative data collected on the radioactivity as to sand, fishes and edible seaweeds. Regarding the elimination in the ground of radioactive slags, at middle and low activity, the author describes construction systems of losing pits and control pits; he illustrates the various factors (absorption, ion exchange, chem. reaction, filtration intervening in the dispersion process, and the existing dangers of eventual contamn. of underground hydric beds, he hints at the different diffusibility of underground radioactive elements and at the use of ruthenium-106 and nitrates as detectors of imminent contamn. of deeplying waters and at surface, around the dispersion fields; he illustrates, finally, the systems followed by the Oak Ridge National Laboratory (ORNL) and the Hanford Workshops, regarding the dispersion process in the underground.—*PHEA*

Soil Disposal of Radioactive Wastes at ORNL: Criteria and Techniques of Site Selection and Monitoring. K. E. Cowser & F. L. Parker. *Health Physics*, 1:152 ('58). The ability of soil to remove and retain the bulk of the fission product cations from aqueous soln. is used as the basis for disposal of large vols. of intermediate level wates. An exptl. system consisting of three 1-mil gal disposal pits has been developed at ORNL and through Dec. '56, received a tot. of 5.6 mil gal of waste containing 58,000 c of Cs^{137}, 15,000 c of Ru^{106} and lesser amount of Sr^{90}, Sr^{90}, Co^{60}, Sb^{125} and the rare earths. Sodium ion and nitrate ion account for the largest part of the chem. constituents. The geologic and hydrologic characteristics of the reservation served as a guide in the selection of small areas for detailed study. The information obtained from field studies of a specific area and lab. investns. of the interaction of waste and typical soil were used to estimate the operating characteristics of a disposal pit. After the pit was in operation, the underground movement of the chem. constituents and radionucleides in the waste was followed by sampling and radiologging observation wells and sampling at stream-gaging points in the pit area. These

results have confirmed the preoperating estimate of the direction and path of waste movement and the seepage of anion materials such as Ru^{106} and NO_3^- through the soil.—*PHEA*

Determination of Radioisotopes in Water, Food, and Sewage. R. HAHN & C. STRAUB. *Public Works,* 89:111 (Dec. '58). Radioactive fallout from nuclear weapons, increasing use of isotopes, and failure of a nuclear reactor all create possibilities for the contamination of food and water. Since toxicity of radioactive substances varies greatly between isotopes, and MPC's of various isotopes is available in National Bureau of Standards Handbook 52, the development of rapid and accurate methods of analysis is subject of intensive research. Such studies are being carried on at the Robert A. Taft San. Eng. Center and the chemistry lab. of Wayne State Univ. Methods for water analysis, sewage analysis, and food analysis are mentioned. The latter two require destruction of organic matter, after which water analysis methods may be used. Sepn. using

carrier precipitation is often required. Concn. by pptn. or ion exchange may be resorted to.—*PHEA*

Estimating the Uptake of Radioactivity in Streams. E. K. HARRIS. *Sew. & Ind. Wastes,* 31:7A ('59). Assuming a mixt. of radionuclides emitted into a stream, the proportion of activity lost through uptake or deposition between two downstream sampling stations may be estimated by comparing observed activity densities at such points with those expected from natural decay alone. The latter are estimated from a mathematical model fitted to decay factors measured in water representative of the outfall. Selective uptake or deposition which alters the relative concns. of the nuclides significantly may be detected by comparng initial average decay rates observed at downstream stations with avg. decay rates predicted for these stations from the model fitted to the outfall sample. Data from a survey of radioactivity in the Columbia R. during '51-'53 are used to illustrate the procedures.—*WPA*

HYDROLOGY, CONSERVA-
TION & IRRIGATION

Measurement of Ground Water Flow.
P. DANEL. *Tech. l'Eau* (Brussels), 12:133:
21 ('58). The author outlines the basic prin-
ciples underlying the flow of liquids through
porous media and describes methods for de-
termining the rate of flow of ground water
involving the use of dissolved salts or color-
ing matter.—*WPA*

**Use of Radioactive Isotopes in Hydrol-
ogy.** H. MOSER & F. NEUMAIER. *Gas- u.
Wasserfach.* (Ger.), 99:324 ('58). The
authors summarize two articles published by
them [*Atomkernenergie*, 1:26 ('58) and 2:
225 ('57)]. The first deals with the detn.
of amt. of water entering and leaving given
stretches of water courses. A measured amt.
of a radioactive isotope (iodine-131) is
added above a measured stretch and its con-
tent is detd. at intervals at the beginning and
end of the stretch. The use of the method
in different conditions of addn. and with-
drawal of water, the choice of a suitable
isotope, and details of procedure are de-
scribed. The second article describes the
use of radioactive isotopes for measuring the
filtration rate of ground water entering a
well filter and application of the measure-
ment to detn. of the flow of ground water.
—*WPA*

**Use of Radioisotopes in Studies of
Ground Water Flow—Laboratory and
Field Experiments on the Suitability of
Various Tracers.** E. HALEVY ET AL.
Intern. J. Appl. Radiation & Isotopes, 3:293
('58). Abstract of paper presented at the
2nd United Nations International Conference
on the Peaceful Uses of Atomic Energy,
Geneva, Sept. '58. Lab. expts. were carried
out on the suitability of various radioactive
isotopes for ground water tracing. The iso-
topes used were iodine-131, zinc-65, chlo-
rine-36, and iridium-192. A small-scale field
expt. was also carried out with iodine-131
to trace the flow of ground water between
2 wells, 50 m apart; the results permitted
the determination of some hydrological char-
acteristics of the aquifer.—*WPA*
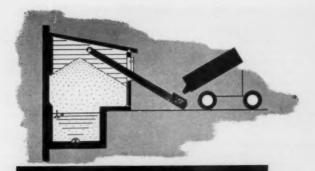
**Ground Water Studies With Radioactive
Hydrogen (Tritium).** H. V. BUTTLAR &
I. WENDT. *Intern. J. Appl. Radiation &
Isotopes*, 3:294 ('58). Abstract of paper pre-
sented at the Second United Nations Inter-
national Conference on the Peaceful Uses of
Atomic Energy, Geneva, Sep. '58. It has
been observed that the concn. of tritium in
rainwater increases considerably after the
explosion of a hydrogen bomb, and then de-
creases again over a period of 40 days.
Corresponding increases in the tritium con-
tent of ground water also occur, but with a
certain time lag of the peak activity in rela-
tion to the activity of the rain water. From
this, it is possible to calculate the rate of
flow of ground water from the area of pre-
cipitation to the sampling point. This
method has been used to measure the move-
ment of ground water in a valley near
Carizozo, N.M.—*WPA*

**Calculations of Reductions in Level of
Ground Water With Reference to Ground
Water Resources.** H. SIEMON. *Deut.
Gewaesserkundl. Mitt.* (Ger.), 2:63 ('58).
Description is given of methods for calculat-
ing the effect of processes for reducing the
level of ground water with greater accuracy
than is obtained by the methods generally
used. Not only the actual reduction but the
time required, the extent of the influence, and
local conditions are considered.—*WPA*

**Experiments on the Flow of Water
Through Beds of Spherical Particles.** E.
INDRI. *Acqua n. Agric.* (It.), 36:65 ('58).
After a brief review of literature on the the-
ory of filtration through porous media, appa-
ratus and expts. are described in which the
nature of the filter medium could be varied
by packing spherical grains uniformly or
irregularly, and the Reynolds number varied
in the range 0.02 to 2,000. The results are
shown graphically and discussed with special
reference to the effect of the method of set-
tling the particles, giving different "void
indexes"; evaluation of the mean diameter
representing each mixed settling; and transi-
tion from laminar to turbulent flow.—*WPA*

**Movement of Dissolved Oxygen Through
Sea Water.** E. NELSON; R. MOORE; &
W. QUINN. *Corrosion*, 14:351t ('58).
Expts. in connection with the mechanism of
corrosion in sea water have shown that,
after deaerated sea water had been allowed
to stand for some time, the concn. of oxygen
was uniform throughout most of the water,
and it is therefore concluded that convection
is more important than diffusion in the

movement of dissolved oxygen through sea water. If diffusion was significant, there should have been more oxygen near the surface of the water than at greater depths. It is pointed out that in many corroding systems there would be more liquid agitation than in these expts., and certain corrosion patterns that have been attributed to an oxygen concn. gradient in the liquid may have been caused by other factors.—*WPA*

Effect of River Channel Roughness on the Velocity of Flow. J. MARTINEC. *Hydraul. Res. Inst.* (Czech.), Paper 96 ('58). Existing formulas for detg. the mean velocity of flow in open channels are discussed; none of the formulas is of general application. Difficulties in the use of such formulas are considered, especially those involved in choosing the appropriate roughness coefficient. The effects of river channel roughness on velocity of flow were investigated by measurements in selected reaches of some river in Czech., and it was found that the differences in river channel roughness in natural reaches are often considerable, even when they have a similar appearance. The formulas do not take these differences into account, and the detn. of roughness coefficients from tabd. classifications of channels is also very difficult, since it depends on a subjective examn. of the stream. The avg. height of bed roughness was found to be useful in investigating the roughness coefficient. Some beds, such as those consisting of sand and gravel, can best be characterized by sieve analysis to give a percentage mass curve of size distribution; by this method good results were obtained in both natural and regulated streams. It is emphasized that the velocity of flow is affected by the type of roughness as well as the height of bed roughness. A formula is presented for use when it is possible to measure or at least to est. the avg. height of bed roughness. In this formula the effect of roughness is expressed by a factor of relative roughness R/d, where R is the hydraulic radius and d the avg. height of bed roughness (in either meters or feet). The constant involved in this equa-

tion was determined exptlly. for various ranges of R, d, and J (the slope of the water surface).—*WPA*

DISINFECTION

Ultraviolet Radiation for Water Treatment. F .R. STIFF. *Chem. Prods.* (Gr. Br.), 22:123 ('59). A brief account is presented of some recent advances in the use of ultraviolet radiation for the treatment of industrial water, e.g., for sterilizing water used in pharmaceutical prepns., for preventing bacterial slime, and for reducing the yeast count in cider and beer.—*CA*

A New Apparatus for the Disinfection of Water With Bactericidal Rays. A. A. BOIKO. *Voenno-Med. Zhur.* (USSR), 4:81 ('58). *Abstracts of Soviet Medicine*, 3:3: B1203 ('59). Lab. expts. with different models resulted in the choice of a device with submerged radiation sources. The device consists of 3 main parts : (1) a section for the radiation of water, (2) a tissue filter and (3) a starting device for the ignition of the bactericidal lamps. Bactericidal lamps BUF-60 were used for ultraviolet radiation.—*PHEA*

Sterilization of H₂O by O₃. P. GUINVARC'H. *Advances in Chem. Ser.*, 21:416 ('59). Filtered water at Saint-Maur is treated with O₃ so that the column's residual concn. is 0.10 ppm below 10°C and 0.05 ppm above 10°C. Dosage fluctuates between 0.6 and 1.5 ppm and avg. treatment is 1.1 ppm. Treated water never shows *Escherichia coli*, and *Clostridium perfringens* is reduced by 50% that of the filtered H₂O. Org. matter, color, and taste are reduced, and low dosages of phenol are destroyed. Energy consumption and cost are also discussed.—*CA*

Development of European Ozonation Techniques. P. FRISON. *Advances in Chem. Ser.*, 21:443 ('59). The historical development of ozonators and O₃ production in Europe is reviewed. O₃ is used mainly for the sterilization of H₂O. From this aspect the processes comprise 3 operationsl O₃ production, conditioning of the air prior to elec. discharge, and contact between O₃ and H₂O. The Otto, van der Made, and Siemens processes for production are reviewed. Recent improvements are indicated for production, conditioning, and contacting.—*CA*

Ozone Effectiveness, Production, and Cost in Water Treatment. E. L. BEAN. *Advances in Chem. Ser.*, 21:430 ('59). The engineering design and capital cost of a plant using O₃ generating units is described. It was detd. that equip. to produce completely dust-free and thoroughly dried air proved valuable. Maintenance cost has been greatly below that expected, while operational costs are not inconsistent with costs for H₂O treatment with other oxidn. processes. Dosage of O₃ is controlled by no. of generators in operation. Ozonation proved effective on removal of material causing taste and odor, particularly the phenolics in the Schuylkill R. Kills of coliform organisms and of bacteria which grow at 37° on agar were approx. 90.5% for avg. O₃ residual of 0.06 ppm, 94.5% for 0.08 ppm, and 98% for 0.16 and 0.20 ppm. With O₃ residuals of 0.20, 0.16, and 0.20 ppm the Mn removal was, resp., 78.2, 82.5, and 80.0%.—*CA*

A Comparison of Ozonation Methods for Drinking-Water Disinfection. A. TORRICELLI. *Advances in Chem. Ser.*, 21:453 ('59). A comparison of conventional means of ozonation, i.e., the Otto and the Van der Made processes, and 2 new methods, a process used by Welsbach Corp. and the Torricelli method showed these newer methods permit more effective and economical utilization of O₃. Data and operating experiences with pilot plants utilizing these systems are presented.—*CA*

Ozonations of Nice's Water Supply. H. LEBOUT. *Advances in Chem. Ser.*, 21:450 ('59). Nice uses filtered H₂O from the La Vesubie R., finds the treated H₂O is often opalescent, turbid, and always contains pathogenic bacteria and various saprophytic germs. Since '06 Nice has been sterilizing this filtered H₂O by using O₃. A year of operational data are given for the 2 treatment plants.—*CA*

Disinfection of Water and Sewage Containing Ammonia With Halides. G. GAD; A. ALBANO; & M. MANTHEY. *Gesundh.-Ingr.* (Ger.), 80:145 ('59). In desalted water, below pH 4, chloramine does not change to N and Cl. The conversion of chloramine to N and Cl is also dependent on time. Br acts the same way as Cl; I does not react with water.—*CA*

Chlorine in Water Chlorination. J. FUCHS. *Chemiker-Ztg.*, 83:153:5 ('59). The reaction mechanism $Cl_2 + H_2O = H^+ + Cl^- + HOCl$ as well as the formation of ClO_2^- and Cl_2O are discussed. The compns. of pure HOCl solns. and of partly neutralized HOCl solns. for the pH range 4–14 are tabulated.—*CA*

Free Residual Chlorination. M. PERRIA & V. BARBIERI. *Igiene Mod.* (It.), 51:(1/2):87 ('58); *Excerpta Medica, Public Health, Social Medicine & Hygiene*, 5:246 ('59). The principles on which this method is based are reviewed and discussed. In chlorination of certain types of water for drinking use, particularly surface water, it is preferable to other methods. It is suggested that this chlorination procedure be widely employed in Italy, as had already been done in other countries, in view of the fact that in Italy, too, the use of surface water is necessary to cope with the increasing water demand. —*PHEA*

The Biological Effect of Chlorine and Derivatives on Water Disinfection. E. NAUMANN. *Tech. l'Eau* (Brussels), 12:136: 17 ('58). The bactericidal effects of Cl are multiple and detd. by the morphologic and physiol. particularities of the microorganisms. In case of bacteria, ClO^- oxidizes the cell enzymes. The reaction rate depends essentially upon the pH value and the temp. of water. This process is characteristic of all compds. releasing Cl: gaseous Cl, hypochlorites, chloroamines. In the case of viruses, it is their own constituents which are oxidized. Sterilization is, however, only efficient if a preliminary purification of water has removed all compds. able to react with Cl to to protect mechanically the viruses. The various compds. to be avoided are: Fe, Mn, nitrites, S compds., and org. compds. reacting with Cl by addn. or substitution. Several species of algae are resistant to Cl. Chlorination is actually the most frequently used of desinfection methods because of its simplicity and efficiency.—*CA*

Four Years of Practical Experience With Chlorine Dioxide. O. WIDEMANN. *Vom Wasser*, (Ger.), 24:50 ('57). Part of the water supply of Basel is drawn from a ground water stream running almost parallel to the course of the river Wiese. In 1950 poln. by phenol from a damaged sewer

carrying waste waters from a gas works appeared and spread to several of the wells. Further polln. was stopped but it was not until 1955 that all the wells could again be brought into use. Expts. are described on the treatment of the pold. water with ozone and chlorine dioxide. Expts. with ozone showed that the difficulties were great and the amt. of ozone required to remove the phenolic taste and odor was extremely high. Parallel expts. with chlorine dioxide were more satisfactory and the ozone expts. were stopped. Treatment of a water contg. 30 $\mu g/l$ of phenol with 0.07 mg/l of chlorine dioxide produced a treated water in which phenol was not detectable by analysis or by taste or odor. The process and reversible nature of the reaction is discussed. A water contg. 20 $\mu g/l$ of phenol was treated satisfactorily with chlorine dioxide for 2 yr. Expts. on the removal of tastes and odors due to other causes are then described. Works are planned in the neighborhood of Basel in which infiltration of treated surface water is to be used to dam back from the intake a ground water stream which is almost free from oxygen and pold. by oil and chems. Oxidation of this water altered but did not remove the taste. Treatment with active carbon proved satisfactory and is to be used until the infiltration plant is in action. Expts. have also been made on the treatment of Rhine water. Treatment by suprechlorination, ozone, and chlorine dioxide gave variable results but in each case subsequent treatment with active carbon reduced the permanganate demand to less than 6 mg/l, which is the max. permissible concn. in Switzerland. Expts. on the bactericidal action of chlorine dioxide showed that heavy bact. poln. was completely removed within 1 min by addn. of 0.06 mg/l chlorine dioxide; 0.12 mg/l chlorine achieved the same effect only after 20 min and the result was not so reliable. The author then describes the principles and reactions of chlorine dioxide treatment, the plant used, and methods of analysis and control of the process.—*WPA*

Disinfection of Water Contaminated With Poliomyelitis Virus. N. V. RYZHOV & E. V. SHTANNIKOV. *Hyg. & Sanit.* (Moscow), 24:3:19 ('59). Expts. on the disinfection of water contaminated with poliomyelitis virus showed that effective disinfection can be achieved by chlorination for 30 min with 0.5–2 ppm residual chlorine.—*WPA*

We may be a little previous, of course. To date the correlation has been no more specific than on a state and regional basis, and a variety of softening agents, together with chlorine, fluorides, iron, manganese, sodium, potassium, magnesium, calcium, carbonates, sulfates, and nitrates, have already been eliminated as causative factors. The only real clue so far seems to be the indication that the more alkaline the water, the greater the protective effect on human arteries.

What bothers us is the likelihood that, just as someone last month suggested that soft water reduced the possibility of contracting poliomyelitis, next month another expert will indicate a correlation between hard water and high cancer incidence and the

month after that a relationship between "neutral" water and tuberculosis will be found. About the only correlation we are willing to subscribe to at the moment is that between water drinking and mortality. Whether the water be hard or soft, hot or cold, polluted or pure, the mortality rate among its drinkers tends to be high.

Leo Louis, president, Gary-Hobart Water Co., Gary, Ind., has been elected president and director of Long Island Water Corp., Lynbrook, N.Y., to succeed the late W. Victor Weir. He will continue in active management of the Gary utility.

A member of AWWA since 1942, he was nominated by the Indiana Section in 1947 to receive the Fuller Award. He is also a member of ASCE.

A National Conference on Water Pollution has been scheduled for December in Washington by directive of President Eisenhower. A steering committee has been formed, on which AWWA is represented by Morrison B. Cunningham, director of public works at Oklahoma City.

Leo Weaver has been appointed chief of the Water Quality Section of the Water Supply & Pollution Control Div., Robert A. Taft Sanitary Engineering Center. He was formerly assistant director of the American Public Works Assn.

R. Steven Rankin, consultant, Stamford, Conn., died Jun. 6, 1960, at the age of 68. Born at Henry, S.D., he received his degree in civil engineering from the University of Minnesota. From 1914 to 1916, he was employed by the Minneapolis Water Works Dept., after which he joined the firm of Pearse, Greeley & Hansen (now Greeley & Hansen), serving as draftsman, designer, and principal assistant engineer. In 1927, he was employed by Dorr-Oliver, Inc., of Stamford, where he became sales engineer, assistant manager, and manager of the Sanitary Engineering Div. In 1955, he was appointed director of the Sanitary Engineering Technical Div.

A member of AWWA since 1940, he served as manufacturer's representative on the Board of Directors in 1956–59.

He was president of WSWMA in 1954, and a member of other technical organizations, including ASCE and WPCF.

Caleb Mills Saville, consultant to the Hartford (Conn.) Water Bureau, died on Feb. 14, 1960, at the age of 94. Born at Melrose, Mass., in May 1865, he received a degree in engineering from Harvard in 1889, after which he joined a consulting engineer firm. From 1893 to 1895, he was assistant to the city engineer at Malden, Mass., after which he served for ten years as division engineer of the Boston Metropolitan Water Works. After two years (1906–07) as hydraulic engineer with a private firm, he went to the Canal Zone to serve as a consulting hydraulic engineer on the construction of the Panama Canal. He was largely responsible for the supervision of the construction of Gatun Dam. In 1912, he went to Hartford, Conn., to become manager and chief engineer of the Board of Water Commissioners, a position he held until his retirement in 1948. After retirement he was retained by the commission as a consultant.

A member of AWWA since 1916, he was a Life Member and one of the oldest members of the Association.

Employment Information

Positions Available

Hydraulics engineer to serve as Director of Public Utilities in city of 45,000. Present director retiring for age. Starting salary approximately $9,000 per year. Apply to City Manager, City Hall, Oshkosh, Wisconsin.

CIVIL ENGINEER—Sanitary option preferred. Applicant must be well qualified, aggressive, and experienced in planning, design, construction, and operation of water treatment facilities pertaining to numerous ground water supplies on Long Island. Excellent opportunity for a permanent position with a large, expanding authority. Interview arranged on basis of letter stating qualifications, educational background, previous employment and compensation, also salary desired. Write:

SUFFOLK COUNTY WATER AUTHORITY
P.O. Box 37
Oakdale, Long Island, New York

OPERATING VICE-PRESIDENT

Outstanding opportunity for technically qualified administrator required by stockholder-owned, multi-plant water works utility system in East. Capable of assuming full responsibility for direction of local supervisory personnel, including optimum utilization of field operating forces, negotiation of union contracts, engineering and scheduling of construction. Resumes should contain a detailed description of all present and prior responsibilities relevant to the position described, include age and present earnings, be accompanied by a photograph, and state at least three business references. No inquiries of references will be made without permission; and all letters will be treated in utmost confidence. Box 071A, Journal American Water Works Assn., 2 Park Ave., New York 16, N.Y.

Public Health Engineers—Statewide stream pollution control program. Salaries $480–$920. Bachelor's degree in engineering required. Starting salary dependent upon qualifications and experience. Apply: State Board of Health, Capitol Station, Helena, Mont.

Service Lines

Membrane filters are described in an illustrated, two-color, 28-page technical brochure. Equipment designed for all requirements for microfiltration is completely specified in the booklet titled "Millipore," available from Millipore Filter Corp., Bedford, Mass.

Fluoride feeders that operate manually or automatically are described in a 4-page specification sheet. Designed to mix and feed a fluoride solution smoothly, the wall-mounted dispenser is the subject of Specification 70D1100, available from Fischer & Porter Co., 481 Jacksonville Rd., Warminster, Pa.

Coal-tar protective coatings are described fully in a 13-page, illustrated booklet, which includes detailed specifications and information on resistance properties. The coatings, particularly suitable for use on buried or immersed steel and concrete structures, are the subject of Bulletin CAC-11-1958, which may be obtained from Allied Chemical Corp., Plastics & Coal Chemicals Div., Box 75-W, 40 Rector St., New York 6, N.Y.

Coagulant aids are the subject of a new 8-page, illustrated brochure that discusses uses, effectiveness, and methods of feeding these chemicals. Copies of HSP-919 are available by writing Hagan Chemicals & Control, Inc., Hagan Center, Pittsburgh 30, Pa.

Precision-ground gears are described and fully illustrated in a 12-page booklet, "An Advanced Concept in Modern Gears." The information, which includes many photographs, may be obtained by writing (on company letterhead), Philadelphia Gear Corp., Schuylkill Expressway, King of Prussia, Pa.

Index of Advertisers' Products

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Philadelphia Quartz Co.

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Zeolite: see Ion Exchange Materials

Why this "magnetic" meter is always easy to read

In Rockwell Sealed Register meters a powerful magnetic coupling transmits every motion of the measuring chamber directly to a hermetically sealed register.

This construction permits the sealed register to be positioned high and dry above the flowing stream. Condensation and dirt, the bane of the meter reader, just can't occur. In fact, this meter will run under water without harm.

The sealed register assembly contains all the gearing both for register operation and for intermediate reduction. Gears and pinions are of the free running instrument type. They offer only token resistance to the measuring element and so improve the accuracy and response of the meter especially at low flows.

All this, plus a simplified design in which only two parts operate in water means less wear, less chance for corrosive attack and of course easier maintenance at lower cost. Get facts now. Write to Rockwell Manufacturing Company, Pittsburgh 8, Pa.
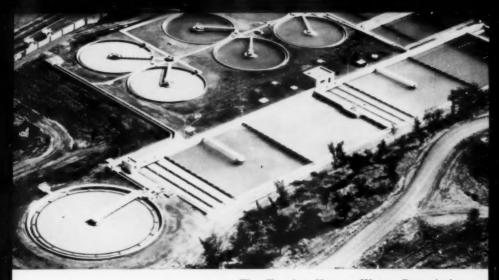
POWERFUL MAGNETIC DRIVE

Illustrating how the driver magnet (held in hand) revolves around a watertight compartment at base of register to push the follower magnet, which in turn drives the register shaft.

SEALED REGISTER METERS

another fine product by

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For the third time...

TOPEKA, KANSAS SPECIFIES DORR-OLIVER WATER TREATMENT EQUIPMENT!

Expanded facilities now ready to meet estimated demand through 1970

The Topeka, Kansas Water Commission has recently completed the second extensive addition to its water treatment facilities on the Kansas River. This time—as with an earlier addition and with the original installation—Dorr-Oliver mixing and clarification equipment was selected. Total capacity of the plant is now 50 mgd —sufficient to meet projected demand for the next decade.

The new Dorr-Oliver additions include: One 130′ diameter by 13′ s.w.d. Type S pre-sedimentation clarifier for removal of silt preceding softening and filtration; one primary and one secondary 120′ long by 45′ wide by 14′6″ w.d. Type T Flocculator; one 120′ square by 15′6″ s.w.d. secondary Squarex clarifier.

The original plant and first addition included seven primary and secondary Dorrco Flocculators, two Squarex clarifiers and four Dorrco flash mixers.

This continuing confidence in Dorr-Oliver units and methods is testimony to the efficiency of D-O equipment, and the reliability of D-O service. For full information write to Dorr-Oliver Incorporated, Stamford, Connecticut.

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